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## THE IRRIGATION PROJECT OF THE CANADIAN PACIFIC RAILWAY.

By a Special Contributor.

The rise in value of farm lands in the western part of the United States during the past ten years has caused them to double and treble in price. This has had two results. It has tempted many farmers to sell and discouraged many others from attempting to buy. Those who have sold and homeseekers wishing for lands within their means are beginning to look elsewhere than the United States, and this is changing the tide of emigration from the West to the North. Canada is becoming the Mecca for the homeseeker of the United States. The 46 emigrants to western Canada eight years ago swelled to 46,000 in 1904. Because irrigated lands in the arid region have risen in value until in many sections they equal or surpass in price the farming lands of Iowa and Illinois, the seeker for cheap lands must go elsewhere, and the picturesque prairie schooner from western Nebraska and Kansas is now finding its slow way for over 2,000 miles, and more than 300 miles north of the northern boundary of the United States, to the settlements in the valley of the Saskatchewan River. The reports which these emigrants are sending back are upsetting all preconceived ideas as to climatic conditions and agricultural possibilities. Settlers from western Nebraska write back from Edmonton that the winters, while longer, are no more disagreeable than those of the home they left in the United States a thousand miles south. It has already been demonstrated that the northern limit of settlement on this continent is to be extended far beyond what was formerly regarded as possible, and that this is destined in time to have a profound influence on the adjustment of centers of manufacture, agriculture, and population on this continent.

GRADER LOADING WAGON ON IRRIGATION CANAL OF CANADIAN PACIFIC RAILWAY NEAR CALGARY, N. W. T.

The results already achieved by these settlers confirm this conclusion. Eighty million bushels of wheat, 50,000,000 bushels of oats, 10,000,000 bushels of barley, and $10,000,000 worth of live-stock were among the items of agricultural production in western Canada in 1904. If the present growth continues, it will only be a few years until the provinces of Alberta, Assiniboia, and Saskatchewan will have a larger acreage and larger yield than the three great hard-wheat States of Minnesota, North and South Dakota. The improvements which are being made by these settlers show that these farmers have gone there to stay. The brick and stone farmhouses, the concrete barns and stables, which one sees scattered throughout the older settled portions of Manitoba are beginning to be built in Assiniboia and Saskatchewan. This growth of settlement in the country is being duplicated in the cities. Winnipeg has now 75,000 people. The Canadian Pacific Railway is erecting a hotel and station which will cost $1,200,000, and the building permits in 1904 aggregated over $10,000,000. The terminal facilities of Port Arthur on Lake Superior, built to accommodate the lake shipments of grain, include the largest elevator in the world.

This movement of settlement towards Canada has been greatly promoted by the Canadian Pacific Railway. It has been a liberal and effective advertiser. It has sold its lands on the most liberal terms and given cheap freight rates to intending settlers. Emigration has now, however, almost absorbed the railway lands which can be cultivated without irrigation and the railroad is now engaged in an immense irrigation enterprise for the purpose of promoting the settlement of that portion of its land grant in which irrigation is required. Originally this grant extended on both sides of the track for a width of 20 or 25 miles, but included only each alternate section. This checkerboard arrangement was not suited to irrigation development and the Government has modified the grant by giving the railroad a solid block of land 25 miles wide along its line between Calgary and Kininvie, a distance of 125 miles. In this block, which contains about 3,000,000 acres, it is planned to irrigate more than 1,000,000 acres by a system of canals taking water from the Bow River. The flow of this stream is ample for the needs of this land, and is unusually uniform because its summer supply is derived largely from glaciers which act as an automatic regulator; the warmer the season and the greater the need of irrigation, the more rapidly the ice melts and the greater the water supply.

When the system is completed, it will include two main canals, one starting near Calgary and the other about 25 miles below. These canals have the first right to the river. Both will be under one control, and as there is ample water to fill them, irrigators will be freed from the uncertainties and controversies about water rights which have been a perplexing fact in western irrigation development.

The map (Fig. 1) shows the boundaries of this block and of the canal lines now being built. The upper canal will irrigate about 300,000 acres, which will be distributed through approximately 1,200,000 acres of the irrigation block. About one acre in four or 40 acres in each 160 will be irrigated. For each 40 acres irrigated, there will be 120 acres to be used as pasturage or cultivated by the aid of rainfall alone. In this way the productive capacity of the whole body of land will be very greatly augmented. Water will be used on the irrigated lands to better advantage and crops will be given more careful cultivation. These grazing lands have considerably greater productive capacity than those of the United States; probably twice as much grass grows on an acre near Calgary as around Helena in Montana, and four or five times as much as in New Mexico and Arizona. Furthermore, a large part of the unirrigated area will in time be cultivated. Experience thus far has shown that it is possible to grow crops on these lands three years in five, and certain crops can be grown every year. By this combination of the grazing and irrigable areas, it is believed that the whole tract can be made as productive and be as thickly settled as an equal area in the Mississippi Valley.

The canal now being built is to water the western or upper end of the tract. The work is being done by the railroad under the direction of Mr. J. S. Dennis, Superintendent of Irrigation, Mr. P. T. Bone being the chief engineer.

The headgate is about a mile from Calgary on the first bottom of the Bow River, where the

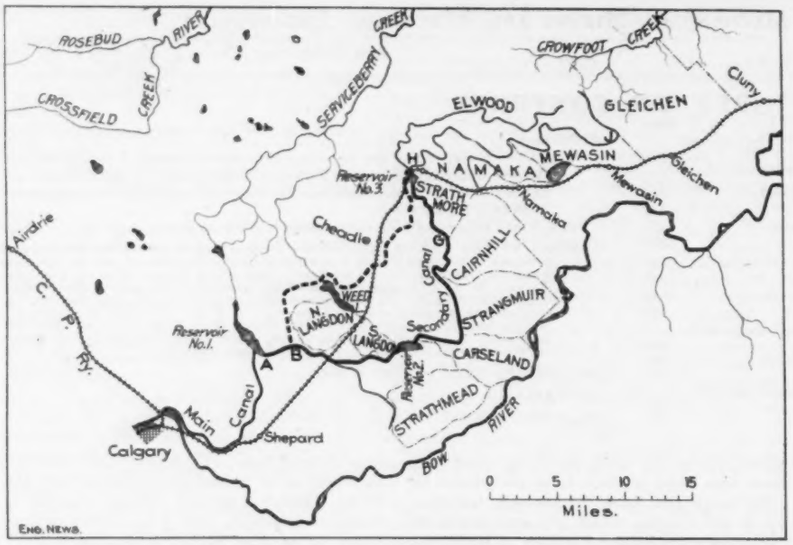



FIG. 1. MAP OF IRRIGATION BLOCK, CANADIAN PACIFIC RAILWAY, NEAR CALGARY, N. W. T.

Sizes of Secondary Canal A.

| Division. | Width, ft. | Depth, ft. | Slopes. | Grade, %. | Velocity, ft. per sec. |
|---|---|---|---|---|---|
| a—b | 18 | 8.0 | 2 to 1 | 0.03 | 3.05 |
| b—c | 18 | 7.5 | " | 0.03 | 2.92 |
| c—d | 18 | 6.5 | " | 0.04 | 3.1 |
| d—e | 22 | 6.0 | " | 0.03 | 2.7 |
| e—f | 18 | 6.0 | " | 0.03 | 2.6 |
| f—g | 18 | 5.0 | " | 0.03 | 2.4 |
| g—h | 18 | 4.0 | " | 0.05 | 2.7 |
| h—i | 16 | 4.0 | " | 0.04 | 2.36 |
| i—j | 12 | 4.0 | " | 0.04 | 2.24 |

banks are low and no excess excavation is required. This bottom is only a few hundred feet wide and back of it is a high steep bluff which follows the river for nearly three miles, where there is a break caused by an old river channel which runs almost at right angles to the present channel which has been formed by a cutoff across a bend. The canal runs on the first bottom to Station 137, where it cuts through a point in order to shorten the distance into the old river channel. The ability to utilize this channel has been of great advantage as it enables the canal to leave the river almost at right angles and reach, within a distance of 20 miles, the summit of the divide between the Bow and Rosebud rivers. This point is the terminus of the main canal and from it secondary canals radiate in three directions.

The bottom width of the main canal is 60 ft.; width on the water line, 120 ft.; side slopes, 3 to 1; depth of water, 10 ft.; sectional area 900 sq. ft. The grade is 0.528 ft. per mile and the computed discharge is over 2,000 cu. ft. per sec.

Between the headgate and Station 137 the location of the canal involved some unusual features. To keep the canal line on grade would have involved throwing it onto the bluff bordering the bottom. As this bluff was steeper than the side slopes of the canal, it would have necessitated a cut reaching entirely to the top and the handling of an immense amount of material. Furthermore, the soil of which the bluff is composed is not entirely satisfactory, and, to avoid cutting into it, it was decided to let this bluff form one bank of the canal and to make the other entirely in fill. Fig. 2 shows in outline a section of the canal near Station 137. The decision to build the outer bank of the canal in embankment was influenced somewhat by the fact that there is a deep cut beginning at Station 137 and the excess of material from this excavation could be used to make a large part of the fill. As the bottom slopes much faster than the canal grade, this outer bank of the canal continues to rise from the headgate to Station 137, being at the lower end 30 ft. high and the depth to the water line, 26 ft. Where the excavation begins, the grade line of the canal will begin again, and it is expected that in a few years this hole in the bottom of the canal will be filled by the deposit of sediment from the river, but until this happens, the outer bank of the canal is practically an earth dam supporting in its deepest point a pressure of 26 ft. of water.

This embankment is being built in the most careful manner. The entire surface soil under it was stripped, but none of the stripping used; only selected material is being placed in this fill. It is being spread in successive layers 10 ins. in depth, rolled with a roller three tons in weight, fitted with exterior bands 4 ins. in depth and 6 ins. apart, in order to insure a perfect union of the different layers. Where the material is wet enough to compact thoroughly without sprinkling this is not required, but whenever it is not moist enough to make a firm ball of earth by the pressure of the hands, sprinkling is required. This is not only specified on this fill but on all the fills along the main canal, its nearness to the main line of the railroad causing the company to exercise unusual precautions to insure its safety. In order to facilitate sprinkling, the contractors have installed a large pumping plant and pipe line running along several miles of the main canal.

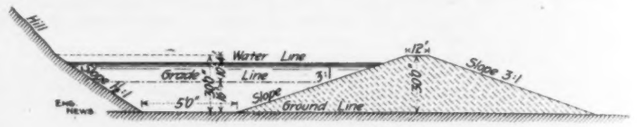



FIG. 2. CANAL CROSS SECTION NEAR STATION 137.

The first heavy cut is 60 ft. deep. In this section of the canal is narrowed to a bottom width of 44 ft., and the side slopes changed to 2 to 1, while the grade is increased to 1.06 ft. per mile, producing a velocity which compensates for the reduced cross section.

The main canal is divided into four sections. On Section 1 there is 714,000 cu. yards, of which 428,000 has been excavated and 286,000 remains to be moved. On Section 2 there is 787,000 cu. yards, of which 323,000 has been excavated and 464,000 remains to be moved. On Section 3 there is 504,000 cu. yards, of which 295,000 has been moved and 209,000 remains. Section 4 contains 351,000 cu. yards. Nothing has been done on this.

The construction of this canal made it necessary to change the main line of the Canadian Pacific Railway for about 4½ miles. This change has, however, resulted in a marked improvement in both grade and alinement.

All of the structures on the canal are of wood. The cheapness of timber and the high price and scarcity of labor needed if masonry or concrete structures were built made it unquestionably economical to use wood. The details of the headgate are shown by Fig. 3 and front and rear views by Figs. 4 and 5. The protection of the river bank is also shown in Fig. 4. The headgate has 30 openings, each 3 × 10 ft. It is set in a foundation of coarse gravel. This is good material except for the danger from excessive leakage under the gates. To prevent this, three rows of sheet piling 4 ins. thick have been driven as deep as possible.

At the first spillway the grade line of the canal is about 14 ft. above the ground and the waste water is let down a plank flume supported by piles. Its construction is shown by Figs. 6 and 7.

Modern machinery is being largely used in the excavation. A floating dredge is used on a part of the main line, and steam shovels, self-dumping carts, and New Era graders are largely used along the entire line in the removal of the surface layers. One of the graders is shown on the first page of this article.

The main canal discharges into a lake whose

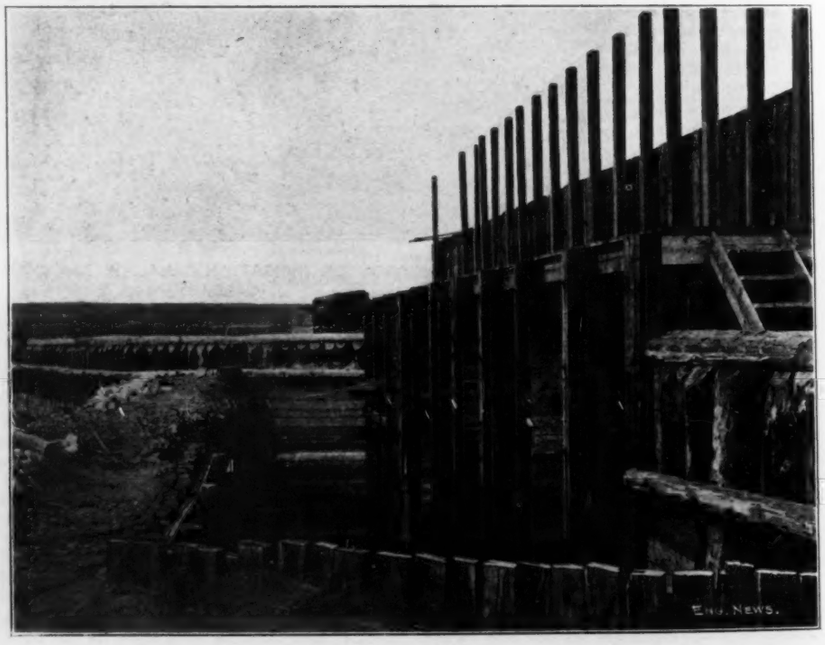



FIG. 4. FRONT VIEW OF IRRIGATION HEAD GATE, SHOWING PORTION OF RIVER PROTECTION WORK.
(Cofferdam to be removed.)

depth will be raised 18½ ft. by a dam at the western end. This reservoir is about three miles long and nearly a half mile wide, having a flooded area of 700 acres. It is a valuable feature of the canal system for two reasons: it will warm the water before it is used in irrigation and serve as a settling basin for sediment and save much work in cleaning out secondary canals.

## SECONDARY CANALS.

The location of the secondary canals in this system is a difficult problem. The country is not, on the plains east of the Rocky Mts., a practically unbroken decline in one direction, but is a series of slopes running towards the streams which traverse this block and interrupted by irregular detached ridges. Not all of the land can be irrigated, and of the land which can be irrigated there is a wide difference in values. Furthermore, the plan of the company to combine the irrigated and unirrigated areas renders it necessary that the irrigated tracts should be distributed throughout the entire block. Hence, in locating the secondary canals, the engineer has to decide three questions: first, the land to be irrigated; next, how far natural channels can be utilized; and third, the number and location of the secondary canals.

In the case of the secondary canal shown on the map there were the following alternatives:

(1.) A main canal from A to B, carrying the full supply of water for all the district.

(2.) A branch canal from B around the north end of Weed Lake, which would divide at Reservoir No. 3, continuing as two canals to J.

(3.) A branch canal from B to G carrying water for the lands south of the railway on the Bow River slope.

(4.) A single main canal from A to Reservoir No. 3, and two canals from there to J, carrying water for all the lands under consideration.

The first three form one plan; the fourth, another. The following figures show there is little difference in first cost. The first cost of the canal under the first plan would be $388,580. Of this, $55,883 would be for timber structures, which it is estimated would have an annual depreciation of at least 7% or $3,912, being 6% interest on $65,200. Adding the capitalized value of this depreciation to the first cost, $388,580, would make the ultimate cost $453,780. The estimated first cost according to Plan 4 would be $395,143. Of this, $38,179 would be for timber work, of which the annual depreciation at 7% would be $2,672 or 6% interest on $44,533, making the ultimate cost of this system $439,676. From this it will be seen that practically the only advantage of Plan 4, so far as ultimate cost is concerned, is the less amount of timber work used. It has been adopted, not so much because of the slight theoretical difference in favor of its cost, because there will be less loss of water and less expense in patrolling if the water is all carried in one canal from A to H. The length of this system will be 65 miles, as against 82½ miles for the other.

This secondary canal will irrigate 110,000 acres, divided into several districts by natural boundaries. The accompanying table gives the areas of these districts, the capacity of the canals which water them, and the cost chargeable to each district for excavation.

To this there will be added an additional charge of $1 an acre for laterals, the surveys of which have not yet been made.

Several of the contractors are from the United States, Winston Brothers of Chicago and Corey Brothers Construction Company of Ogden, Utah, being among the number. About 500,000 acres of the land has already been sold, and it is reported that Minneapolis parties are planning to erect a beet sugar factory in the Gleichen district, at a cost of $800,000.

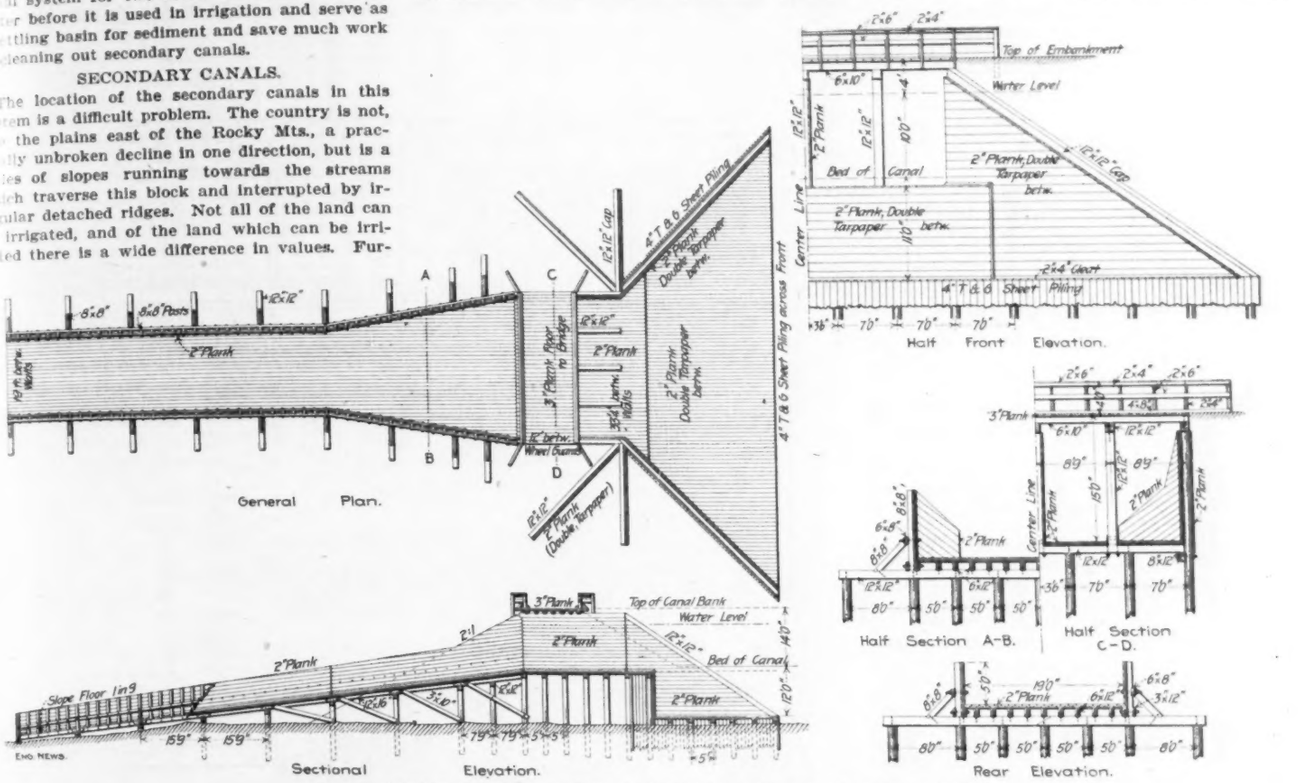



FIG. 3. DETAILS OF HEAD GATE, CANADIAN PACIFIC IRRIGATION WORKS.

Table Giving Estimated Cost for Grading Secondary Canal A and Cost per Acre for Each District Supplied Therefrom.

| Division of canal. | District. | Irrigable acres in district. | Sec. ft. required per District. | Sec. ft. required per Division of canal. | Cost of Canal division. Grading of canal. | Cost of Canal division. Sec. ft. | Cost of District. Sec. ft. | Cost of District. Acre. | Cost of District. Totals. |
|---|---|---|---|---|---|---|---|---|---|
| a — b | North Langdon | 3,400 | 22.67 | 733.33 | $33,674 | $45.92 | $45.92 | .30 | $1,020 |
| b — c | South Langdon | 1,100 | 7.33 | 710.67 | 28,432 | 40.07 | 85.99 | .57 | 627 |
| | Strathmead | 15,000 | 100.00 | ..... | ..... | ..... | ..... | .... | 8,550 |
| c — d | Carseland | 8,800 | 58.67 | 603.34 | 48,240 | 80.00 | 165.99 | 1.10 | 9,680 |
| d — e | Strang Muir | 15,900 | 106.00 | 544.67 | 30,736 | 56.43 | 222.42 | 1.48 | 23,532 |
| e — f | Cairnhill | 16,800 | 112.00 | 438.67 | 43,101 | 98.27 | 320.69 | 2.14 | 35,952 |
| f — g | Strathmore | 6,000 | 40.00 | 326.67 | 22,610 | 69.21 | 389.90 | 2.60 | 15,600 |
| g — h | Namaka | 4,500 | 30.00 | 286.67 | 29,591 | 103.22 | 493.12 | 3.28 | 14,760 |
| | Elwood | 7,000 | 46.67 | ..... | ..... | ..... | ..... | .... | 22,960 |
| h — i | Mewasin | 7,500 | 50.00 | 210.00 | 52,834 | 251 58 | 744.70 | 4.96 | 37,200 |
| i — j | Gleichen | 24,000 | 160.00 | 160.00 | 34,010 | 206.31 | 951.01 | 6.34 | 152,160 |
| | Total areas | 110,030 | | Total cost | $322,228 | | | | $322,041 |

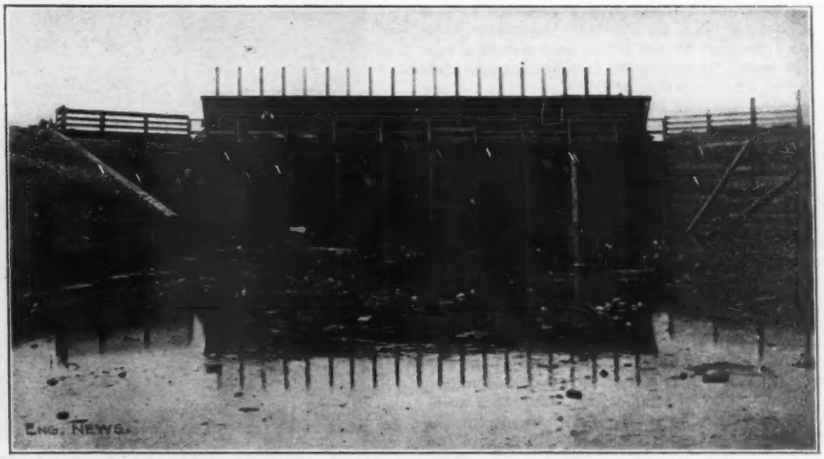



FIG. 5. REAR VIEW OF IRRIGATION HEAD GATE.

## CHARACTERISTICS OF TWO COMPANION STREAMS OF THE PLAINS.

By E. C. Murphy,* Assoc. M. Am. Soc. C. E.

The Neosho and Verdegris Rivers are two companion tributaries of the Arkansas River that drain long and comparatively narrow strips of land in eastern Kansas and Indian Territory. The remarkable fluctuations in the flow of these streams, the maximum being very large and the minimum very small, and the effect of these extreme changes on water power possibilities and losses due to floods along them make a study of their behavior of interest and value.

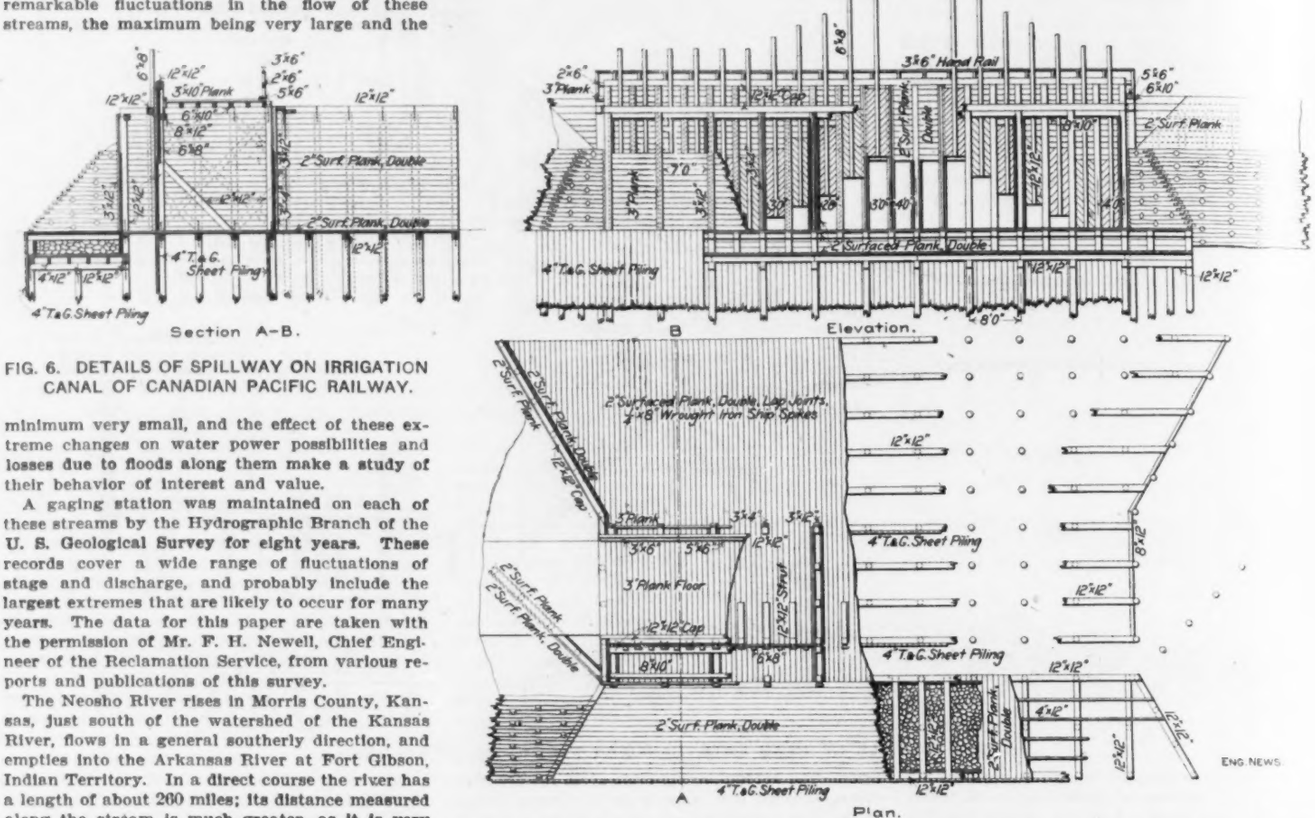



FIG. 6. DETAILS OF SPILLWAY ON IRRIGATION CANAL OF CANADIAN PACIFIC RAILWAY.

A gaging station was maintained on each of these streams by the Hydrographic Branch of the U. S. Geological Survey for eight years. These records cover a wide range of fluctuations of stage and discharge, and probably include the largest extremes that are likely to occur for many years. The data for this paper are taken with the permission of Mr. F. H. Newell, Chief Engineer of the Reclamation Service, from various reports and publications of this survey.

The Neosho River rises in Morris County, Kansas, just south of the watershed of the Kansas River, flows in a general southerly direction, and empties into the Arkansas River at Fort Gibson, Indian Territory. In a direct course the river has a length of about 260 miles; its distance measured along the stream is much greater, as it is very crooked. It falls from an elevation of about 1,500 ft. to 800 ft. at a point 27 miles north of the Kansas and Indian Territory line in 136 miles. From this point to the mouth it falls 300 ft. in about 110 miles. The average fall of the stream from Emporia to the mouth is less than 2 ft. to the mile, and there are stretches of 20 miles or more where the fall is only 1.7 ft. per mile. There are stretches of 10 miles or more where the river is so crooked that the length measured along it is more than twice that measured in a direct course. These numerous bends, together with the trees that grow thickly to the water's edge in some places, reduce the already small slope of the stream so much that the effective slope is probably not more than 1.25 ft. to the mile in many places.

The upper part of the river's drainage area is hilly pasture land, from which the water flows rapidly; the central and lower part is rolling cultivated land. There are no forests on the watershed, but narrow strips of trees are found along the greater part of the stream in Kansas. The principal tributaries are the Cottonwood, which enters it from the west near Emporia, and Spring River, that enters from the east a little south of the Kansas line.

The Verdegris River drains an area immediately west of the Neosho River. As compared with Neosho, the Verdegris rises about 40 miles farther south and at about 100 ft. less elevation, flows nearly parallel to it and empties into the Arkansas only a few hundred feet from the mouth of the Neosho. It falls more rapidly in its upper course than the Neosho, reaching the state line at an elevation of 680 ft., falling from an elevation of 1,400 ft. to 680 ft. in about 151 miles; the Neosho falls less rapidly, reaching the state line at an elevation of about 770 ft. From the state line the Neosho falls about 270 ft. and the Verdegris about 180 ft. As a result of these differences in topography of drainage area, the Verdegris flows in a narrow deep channel; the Neosho is wider, has low banks, and is subject to overflow in many places, notably near Chanute. Near the mouth the differences are very marked; the Verdegris is deep, 250 ft. wide, with scarcely a perceptible current at ordinary stage of water; the Neosho is from 600 to 800 ft. wide, is shallow, and has a fairly rapid velocity.

The principal tributaries of the Verdegris are Fall, Elk and Cany Rivers and Bird Creek; these all enter from the west and flow in a southeasterly direction. The two former have a very rapid run-off, and are important factors in the flood of the main stream.

The drainage area of the Verdegris covers about 8,610 square miles. The upper part is mainly hilly pasture land, having a general fall of about 25 ft. to the mile towards the river; the central portion is general rolling cultivated land. There are no forests in the drainage basin, but occasional narrow strips of trees grow along the streams.

The mean annual rainfall over these drainage basins varies from 35 ins. in the western part to 45 ins. in the eastern part.

The gaging station on the Neosho River is located at the highway bridge at Iola, about 150 miles from the source, and is shown in Fig. 1. Surface fluctuations were read twice daily on a gage from August, 1895, to November 30, 1903,

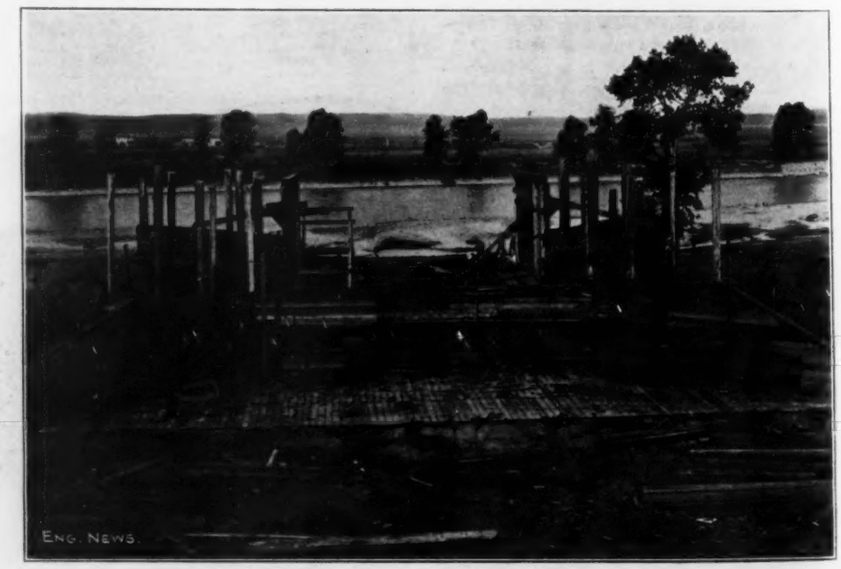



FIG. 7. VIEW OF SPILLWAY UNDER CONSTRUCTION ON IRRIGATION CANAL OF CANADIAN PACIFIC RAILWAY.

*Engineer, U. S. Geological Survey, Hydrographic Branch, Washington, D. C.

TABLE I.—Run-off of Neosho River at Iola, Kansas, for the Period January, 1896, to November, 1903.

| | Cu. ft. per sec. per sq. mile. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1896. | 1897. | 1898. | 1899. | 1900. | 1901. | 1902. | 1903. | Mean. |
| ...n | 0.034 | 0.145 | 0.002 | 0.097 | 0.049 | 0.121 | 0.101 | 0.176 | 0.091 |
| ...b | 0.031 | 0.460 | 0.127 | 0.168 | 0.054 | 0.334 | 0.160 | 0.284 | 0.201 |
| ...r | 0.016 | 0.262 | 0.188 | 0.583 | 0.710 | 0.272 | 0.164 | 0.575 | 0.346 |
| ...r | 0.738 | 0.244 | 0.205 | 0.242 | 0.607 | 1.890 | 0.228 | 0.602 | 0.596 |
| ...y | 2.709 | 0.103 | 2.413 | 0.298 | 1.117 | 0.290 | 1.522 | 2.495 | 1.380 |
| ...e | 0.247 | 0.099 | 1.461 | 1.136 | 0.289 | 0.260 | 3.186 | 2.922 | 1.200 |
| ...y | 0.348 | 0.044 | 0.107 | 0.938 | 0.209 | 0.069 | 0.591 | 0.306 | 0.326 |
| ...g | 0.016 | 0.020 | 0.148 | 0.084 | 0.055 | 0.048 | 1.739 | 1.605 | 0.472 |
| ...p | 0.002 | 0.002 | 0.185 | 0.054 | 0.552 | 0.028 | 1.352 | 0.380 | 0.319 |
| ...t | 0.003 | 0.000 | 0.108 | 0.027 | 0.550 | 0.009 | 1.278 | 0.538 | 0.314 |
| ...v | 0.067 | 0.000 | 0.131 | 0.014 | 0.931 | 0.009 | 0.305 | 1.157 | 0.327 |
| ...c | 0.024 | 0.000 | 0.629 | 0.020 | 0.104 | 0.010 | 0.007 | *0.199 | 0.199 |
| Mean | 0.360 | 0.115 | 0.475 | 0.305 | 0.436 | 0.279 | 0.935 | 0.942 | 0.481 |

*Average of the December run-off for the preceding seven years.

Table II.—Maximum and Minimum Discharge of Neosho River at Iola, Kansas. (3,670 square miles.)

| Year. | Date | Maximum Discharge. Gage Height. | Maximum Discharge. From Total Drainage Area. | Maximum Discharge. Per Square Mile. | Date. | Minimum Discharge. Gage Height. | Minimum Discharge. From Total Drainage Area. | Minimum Discharge. Per Square Mile. |
|---|---|---|---|---|---|---|---|---|
| 1895a | Sept. 12 | 21.00 | ..... | ..... | October | 2.03 | .... | ..... |
| 1896 | May 24 | 21.10 | 45,560 | 12.4 | Sept.-Oct. | 2.00 | .... | ..... |
| 1897 | March 5 | 6.35 | 4,840 | 1.32 | October | 1.70 | .... | ..... |
| 1898 | June 27 | 20.50 | 27,875 | 7.60 | Jan.-Feb. | 1.80 | 1 | 0.00027 |
| 1899 | June 9 | 17.03 | 22,191 | 6.05 | Oct.-Dec. | 2.00 | 50 | 0.014 |
| 1900 | Nov. 2 | 18.95 | 30,411 | 8.29 | Jan. & Aug.-Sept. | 2.10 | 145 | 0.040 |
| 1901 | April 17 | 13.30 | 19,250 | 5.25 | Oct.-Dec. | 0.50 | 10 | 0.0027 |
| 1902 | June 11 | 21.25 | 35,550 | 9.69 | January | 2.00 | 190 | 0.052 |
| 1903 | June 3 | 22.00 | 39,120 | 10.7 | Sept.-Oct. | 2.00 | 120 | 0.033 |
| 1904a | July 10 | 24.00 | 74,600 | 20.3 | ..... | .... | .... | ..... |

(a) Record for part of year.

...nd a sufficient number of discharge measurements made with current meter to enable the ...daily, monthly and annual flow of the stream at ...this place to be computed. The maximum range ...of stage as shown by the gage heights at this ...place is about 22 ft.

The method in use by the Hydrographic Branch of the U. S. Geological Survey for estimating the flow of streams may be found in the Hydrographic Manual, Water Supply and Irrigation Paper No. 95.

The monthly and annual estimates of run-off for the eight years, 1896 to 1903, are given in Table I. Data on the maximum and minimum daily flow of this stream at this place are given in Table II.

Fig. 2 shows graphically the monthly rate of flow at each of these stations, expressed in cu. ft. per sec. per sq. mile. The curve A B is for the Neosho River, and C D for the station on the Verdegris River. Time in months is used as abscissae and rate of flow in second feet per square mile as ordinates.

Although this station was discontinued November, 1903, the gage was not removed, and from high-water marks it was possible to estimate the discharge of the river at this place during the great flood of July, 1904. It is seen from these tables that the maximum yearly flow of this stream may occur in any month from March to July, inclusive, or in November. The largest flood occurred July 10, 1904, and had a mean rate of flow for that day of 74,600 cu. ft. per second, or 20.3 cu. ft. per second per square mile. This flood remained within 4.5 ft. of maximum stage for 11 consecutive days.

It is seen that for two consecutive months in 1896 the river was dry at this place. In 1897 and 1898 it was practically dry for six consecutive months. The daily rate of flow is seen to have varied from 0 to 70,600* cu. ft. per second; the monthly rate of flow varied from nothing to 45,560 cu. ft. per second, and the annual rate of flow varied from 102 to 3,432† cu. ft. per second. These figures, and the curve A B, Fig. 2, very clearly show the flashy character of this stream.

The gaging station on the Verdegris River was located at the highway bridge about six miles south of Independence. Surface fluctuations were observed twice daily on a gage and a sufficient number of discharge measurements made with current meter to enable the daily, monthly and annual run-off to be computed for the period August, 1895, to November, 1903. The maximum range of stage as shown by the gage heights is about 40 ft., that is, about 18 ft. greater than at the station at Iola on the Neosho. The monthly and annual estimates of run-off obtained at this gaging station are given in Table III., and data on the maximum and minimum daily flow in Table IV.

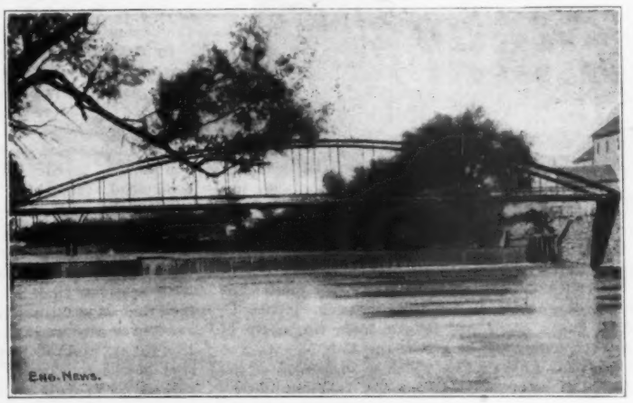



FIG. 1. DAM AND GAGING STATION ON THE NEOSHO RIVER, NEAR IOLA, KAN.

This station was discontinued on November 30, 1903, but from gage readings taken at Independence in 1903-4 during the floods it was possible to compute the maximum rate of flow at this gaging station during the great flood of July, 1904. The rate of flow on July 8, when the stage was a maximum, was found to be about 50,400 cu. ft. per second, or 16.4 second-feet per square mile. From this table it is seen that the daily flow during the period covered by the record has varied from 2 to 50,400 cu. ft. per second; the monthly rate of flow has varied from 2 to 10,864 cu. ft. per second, and the annual rate of flow varied from 556 to 2,030 cu. ft. per second. It is seen that for four consecutive months in 1899 the monthly flow was not more than 18 cu. ft. per second. These results indicate a flashy stream, yet not as flashy as the Neosho River.

It is frequently assumed that the monthly run-off per square mile from two adjacent drainage areas having nearly the same size and topography is practically the same, so that if the monthly run-off from one of them is known, the monthly run-off from the other can be computed. It is difficult to find two adjacent drainage areas that are nearer of the same size and topography than those of the Verdegris and the Neosho. It may be seen from Tables I. and III. that while the mean rate of flow from these basins for the eight years is nearly the same, and the difference is what would be expected from the difference in rainfall, yet the corresponding monthly run-off per square mile is occasionally several times as great from one of these basins as from the other. The mean run-off in 1897 from the Verdegris Basin was more than twice that from the Neosho Basin for the same period. The mean August run-off per square mile for the eight years is seen to be more than three times as great for the Neosho as for the Verdegris basin. This assumption does not, therefore, appear to be a safe one, especially for this section of the country.

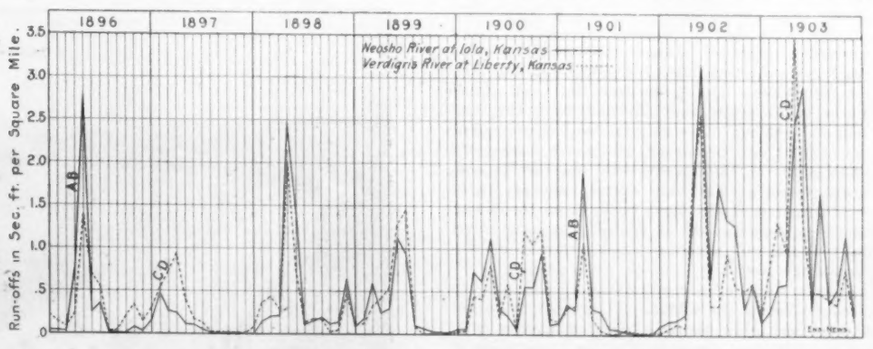



FIG. 2. MONTHLY RATES OF RUN-OFF FROM DRAINAGE BASINS OF NEOSHO AND VERDEGRIS RIVERS.

There is a dam a short distance above the gaging station at Iola, shown in Fig. 1, that has a crest length of 216 ft. and has a head of 8 ft. when the gage reads from 2 to 3 ft. As the river

*Including the year 1904.
†Not including 1903.

TABLE III.—Run-off of Verdigris River at Liberty, Kan., for the Period January, 1896, to November, 1903. *

| | Second-feet per sq. mile. | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1896. | 1897. | 1898. | 1899. | 1900. | 1901. | 1902. | 1903. | Mean. |
| Jan. | 0.216 | 0.289 | 0.035 | 0.104 | 0.021 | 0.151 | 0.015 | 0.243 | 0.134 |
| Feb. | 0.139 | 0.518 | 0.353 | 0.112 | 0.027 | 0.300 | 0.066 | 0.765 | 0.285 |
| Mar. | 0.079 | 0.718 | 0.435 | 0.325 | 0.431 | 0.328 | 0.116 | 1.340 | 0.472 |
| Apr. | 0.221 | 0.923 | 0.280 | 0.406 | 0.403 | 1.067 | 0.087 | 0.987 | 0.547 |
| May. | 1.363 | 0.411 | 2.017 | 0.532 | 0.800 | 0.181 | 1.758 | 3.542 | 1.326 |
| June. | 0.717 | 0.165 | 0.800 | 1.269 | 0.193 | 0.040 | 2.573 | 1.298 | 0.882 |
| July. | 0.554 | 0.133 | 0.127 | 1.445 | 0.594 | 0.004 | 0.342 | 0.505 | 0.463 |
| Aug. | 0.015 | 0.009 | 0.162 | 0.076 | 0.017 | 0.010 | 0.343 | 0.493 | 0.141 |
| Sept. | 0.023 | 0.007 | 0.168 | 0.005 | 1.211 | 0.050 | 0.957 | 0.425 | 0.356 |
| Oct. | 0.196 | 0.003 | 0.020 | 0.002 | 1.055 | 0.018 | 0.556 | 0.358 | 0.276 |
| Nov. | 0.331 | 0.001 | 0.040 | 0.004 | 1.228 | 0.012 | 0.521 | 0.771 | 0.364 |
| Dec. | 0.146 | 0.004 | 0.475 | 0.006 | 0.173 | 0.017 | 0.603 | *0.203 | 0.203 |
| Mean | 0.333 | 0.267 | 0.409 | 0.357 | 0.513 | 0.182 | 0.661 | 0.911 | 0.454 |

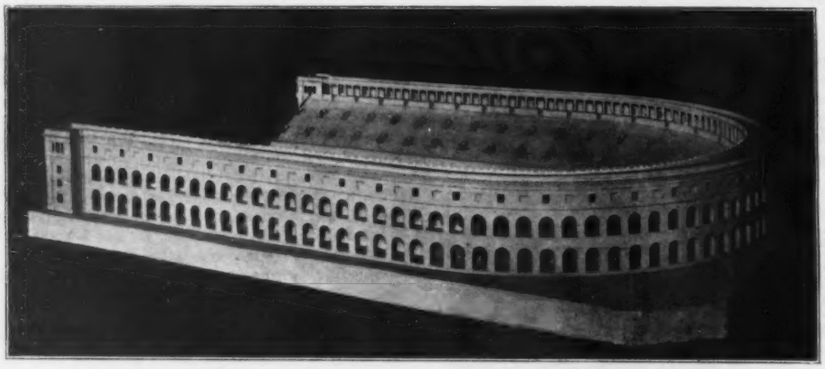
*Average of the December run-off for the preceding seven years.

Table IV.—Maximum and Minimum Discharge of Verdegris River at Liberty, Kansas. (3,067 square miles.)

| Year. | Date. | Maximum Discharge in Second-feet. Gage Height. | Maximum Discharge in Second-feet. From Total Drainage Area. | Maximum Discharge in Second-feet. Per Square Mile. | Date. | Minimum Discharge in Second-feet. Gage Height. | Minimum Discharge in Second-feet. From Total Drainage Area. | Minimum Discharge in Second-feet. Per Square Mile. |
|---|---|---|---|---|---|---|---|---|
| 1895d | Sept. 11 | 35.50a | 37,000 | 12.1 | October | 2.20 | 155 | 0.051 |
| 1896 | May 24 | 23.75 | 19,900 | 6.49 | September | 1.70 | 5 | 0.0016 |
| 1897 | April 30 | 28.00b | 26,100 | 8.51 | Nov.-Dec. | 1.60 | 2 | 0.00065 |
| 1898 | May 4 | 31.00c | 30,690 | 10.0 | September | 1.60 | 2 | 0.00065 |
| 1899 | July 9 | 30.45 | 29,876 | 9.74 | October | 1.60 | 2 | 0.00065 |
| 1900 | Sept. 30 | 36.00 | 36,950 | 12.0 | August | 1.55 | 7 | 0.0023 |
| 1901 | April 13 | 20.40 | 16,260 | 5.30 | Aug.-Sept. | 1.20 | 2 | 0.00065 |
| 1902 | June 7 | 33.37 | 33,018 | 10.8 | Jan.-Feb. | 1.90 | 40 | 0.013 |
| 1903 | May 23 | 39.00 | 41,450 | 13.5 | September | 2.80 | 280 | 0.091 |
| 1904d | July 8 | 41.00 | 50,400 | 16.4 | ..... | .... | .... | ..... |

(a) Absolute maximum, 36.40.
(b) Absolute maximum, 28.20.
(c) Absolute maximum, 31.50.
(d) Record is for part of year.

stage increases, the head on the dam decreases, i. e., the water rises more rapidly below the dam than above it, producing a decrease of head with increase of stage. When the gage reads 9 or 10 ft. the head is reduced to about 2 ft., and it becomes necessary for the mill to shut down for this and all higher stages. During the year 1897 there were 118 days, i. e., nearly one-third of the year, that there was not sufficient water flowing in the river to operate this mill. In 1896 there was sufficient water flowing in the river to run the mill every day of the year; but for nine days the mill was necessarily idle on account of back water.



PHOTOGRAPH OF MODEL OF HARVARD STADIUM EXHIBITED AT ST. LOUIS.

These streams are not as largely used for power purposes at present as formerly, chiefly on account of a better knowledge of the flashy character of the streams. The mill near the gaging station at Iola was burned a few years ago, and has not been rebuilt. The mill near the gaging station on the Verdigris was abandoned some years ago, and the dam is gradually being destroyed by floods. Several other power sites along these rivers have been abandoned within the last few years.

The destruction of property resulting from the overflows of these streams is each year becoming larger as the country becomes more thickly settled. The bottom land along the river is very productive, and large crops of wheat and corn are grown on it. It is subject to overflow for a width varying from one-half mile to three miles. How to regulate the flow of these streams so as to increase the minimum flow for power and city supply purposes and to hold back the peak of the floods so as to prevent overflow is a very important problem. The most natural solution, and the popular one, is storage, but unfortunately the topography of this region will not permit economic storage. The only remedy appears to be the hastening of the flood water from the drainage area, and the use of levees to prevent overflow where the benefits derived therefrom will justify the cost.

Levees have been used along the Neosho in Neosho County since 1892. At present there are 19 of these levee districts, varying in size from 220 to 2,000 acres, depending on the topography of the country. The cost per acre of reclaiming the land has varied from $8 to $12. The flood of July, 1904, overflowed all these levees and damaged some of them to a large extent. They have protected the land, however, against all floods, except the larger ones. By placing the levees at a greater distance apart, so as to give ample water way to the river, making them somewhat higher and protecting them at exposed places, there appears to be no reason except that of cost why the lands along these streams cannot be saved from overflow.

## DETAILS OF REINFORCED CONCRETE WORK IN THE HARVARD STADIUM.

### DESCRIPTION OF STRUCTURE.

The new stadium, or amphitheater for athletic contests, recently built at Harvard University, merits attention from engineers as an example of the use of reinforced concrete on an exceptionally large scale. As its name indicates, the new amphitheater at Harvard is similar to the ancient stadia of Greece. These structures were originally nothing more than a natural hollow, in the basin of which the contests took place, while the spectators were seated on the surrounding sloping hillsides. Later the natural amphitheater was brought artificially into more regular shape and still later the sloping sides were provided with seats of masonry constructed on the ground. The final development was an entirely artificial structure of masonry surrounding a level arena, and it is this last form of structure that is exemplified by the stadium constructed at Harvard.

GENERAL CONSTRUCTION.—In plan the Harvard stadium has the form of a letter U, which has a developed length from tip to tip of 1,390 ft. The axial length of the U is 576 ft., and its width is 420 ft. The width of each arm of the U is 98 ft. The seating capacity is about 25,000. The general character of this structure is quite clearly shown by the cross-section given in Fig. 1. It consists of an inside and an outside wall, parallel to each other and 93 ft. apart, c. to c., and of three intermediate rows of columns and girders parallel to the walls. The inside wall and the three rows of columns support transverse steel girders inclined to support the sloping banks of seats, while the outside wall and the adjacent row of columns carrying two promenade floors running entirely around the seating space, and will ultimately carry a colonnade and roof over the upper of these promenades. With the exception of the inclined transverse girders and their trussed supports, the entire structure is of reinforced concrete. The reinforced concrete construction comprises therefore, columns, girders, curtain and parapet walls, and the floor and seat slabs and stairways. The more important of these structural elements demand separate consideration. The dotted portion of Fig. 1 shows the portion of the structure not yet built.

THE HARVARD STADIUM NEARLY COMPLETED.

COLUMN CONSTRUCTION.—The main supporting elements of the stadium structure are rows of columns carrying lines of girders. There are, as shown by Fig. 1, five of these rows of columns, two of which are within the outside and inside walls. The inside wall columns (Row A) are square and solid and are connected by a solid curtain wall, and the outside wall columns are of special hollow construction which requires separate description. The intermediate columns are all solid. The solid columns vary in size from 12 × 12 ins. to 24 × 33 ins., and each is reinforced by four vertical rods placed at the corners and tied together at intervals by hoop rods. The vertical rods are ⅜-in. for the smaller columns and ½-in. for the larger and the hoop rods are all ¼-in. Further details of the column construction are shown by the accompanying drawings.

In designing the solid columns the reinforcement was counted on to take flexure and such slight shear as might result from overload, but it was not considered as taking any compression. The cross-section was determined by applying an allowed compression stress of 350 lbs. to 400 lbs. per sq. in. to the maximum combined live and dead load, increasing the results thus obtained whenever necessary to keep the ratio of the length to the least side down to about twelve.

GIRDERS.—The reinforced concrete beam work comprises simple girders of 24 ft. 9 ins. span, with sections 16 × 47 ins., 18 × 45 ins., 22 × 60 ins., and 24 × 60 ins., and two systems of curved cantilever girders in Row D, Fig. 1, with sections 18 × 45 ins., and 24 × 60 ins., respectively. In the design of the reinforced concrete girders it was assumed, in providing for flexure: (1) That cross-sections of the beams remain plane after loading, (2) that the tensile strength of the concrete is zero; (3) that the distribution of the stress on the compression side of the vertical axis is parabolic;* (4) that the axis of the parabola is parallel to the longitudinal axis of the beam and its vertex is at the compressed edge*; (5) that the ratio of the modulus of elasticity for steel in tension, $E_s$, to that of the concrete in compression, $E_c$, is 7½*; (6) that the proper limiting stresses for concrete in compression and steel in tension are 500 lbs. and 16,000 lbs. per sq. in., respectively.

The first four of these assumptions, with the aid of the ordinary principles of statics, yield for the moment of resistance of a beam $M = K b h^2$, where b and h are the breadth and depth of the

*Assumptions 3 and 4 are those of Professor Hatt in Engineering News of July 17, 1902; assumption 5 is that of Mr. J. W. Schaub in Engineering News of April 30, 1903.

...the latter being measured to the center ...gravity of the steel reinforcement, K is ...efficient depending upon the ratio, p of the ... reinforcement to the product b h, upon the ...wable working stresses, upon the assumed ...s distributed and upon the ratio of Es to ... and adding a small amount to h for embedding ... protecting the rods, the actual over-all depth ...he beam is found.

...pressing K algebraically, it may be said to be ...hever proves smaller of the two expressions,

$$K = \tfrac{2}{3} f_c \, x \, (1 - \frac{3x}{8})$$

$$K = f_s \, p \, (1 - \frac{3x}{8}).$$

where $f_c$ and $f_s$ are the maximum allowable unit ...stresses in the concrete and the steel, respectively, ...d where x is the fraction of h which measures

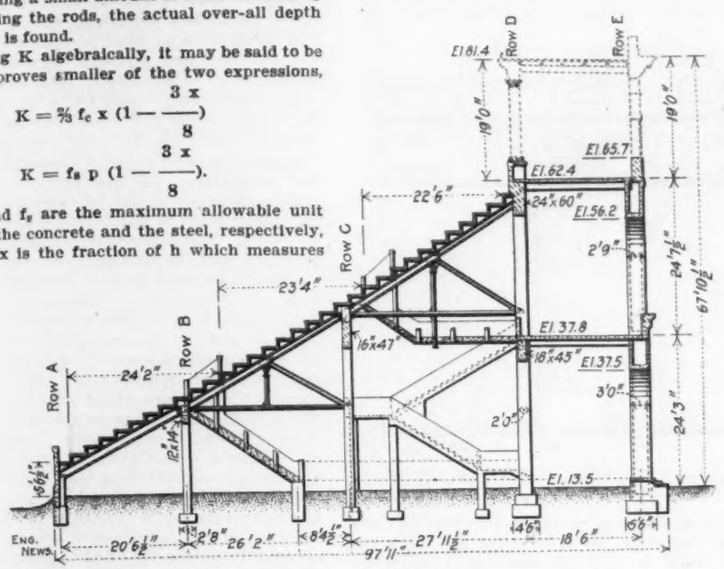



FIG. 1. TRANSVERSE SECTION THROUGH TANGENT OF HARVARD STADIUM BETWEEN AISLES.

the distance from the compression edge to the neutral axis and is written

$$x = -\frac{3}{4} p \frac{E_s}{E_c} + \sqrt{\frac{3}{2} \frac{E_s}{E_c} p \left(1 + \frac{3}{8} \frac{E_s}{E_c} p\right)}$$

The K's were computed and tabulated for various values of p on the basis of assumption 4, 5 and 6. They are given in the Table I in which are

Table I.—Showing Values of K for Concrete and Steel Stresses not Exceeding 500 lbs. and 16,000 lbs. Respectively.

| P. | K for $\frac{E_s}{E_c} = 6$ | K for $\frac{E_s}{E_c} = 7.5$ | K for $\frac{E_s}{E_c} = 10$ |
|---|---|---|---|
| .001 | 16 | 15 | 15 |
| .002 | 31 | 30 | 30 |
| .003 | 45 | 45 | 44 |
| .004 | 54 | 60 | 59 |
| .005 | 59 | 65 | 73 |
| .006 | 64 | 69 | 78 |
| .007 | 68 | 74 | 82 |
| .008 | 72 | 78 | 86 |
| .009 | 75 | 81 | 90 |
| .010 | 78 | 84 | 94 |
| .012 | 83 | 90 | 100 |
| .014 | 88 | 96 | 105 |
| .016 | 93 | 100 | 110 |
| .018 | 96 | 104 | 114 |
| .020 | 100 | 107 | 118 |
| .030 | 114 | 122 | 132 |
| .040 | 124 | 132 | 142 |
| .050 | 131 | 139 | 150 |

included for comparison results for $\frac{E_s}{E_c} = 6$ and 10.

Under ordinary circumstances, p = 0.007 was taken as the most economical value. This gives K = 74 and M = 74 b h². Where lack of room forbade the use of beams as large as this would require, a higher value of p and K indicated a smaller and equally strong beam. On the other hand, where beam sections were fixed by *a priori* conditions at abnormally large sizes, a reduced value of p would be selected and economy of steel effected. Thus it appears that these tabulated K's render the design of reinforced concrete beams for a given bending moment as simple and expeditious as the design of rectangular wooden beams. The K's are seen to be quasifiber-stresses. Multiplied by the product of area and length expressed in b h² they give the moment of a couple as they should. The K's multiplied by 6 give quantities comparable to the fiber stresses allowable in wooden beams. It will be observed that reinforced concrete beams are usually to be rated as inferior in strength to spruce ones of the same dimensions, and this, too, apart from questions of their greater overweight. The following is a numerical example of the use of the K's:

Problem.—Required the size and reinforcement of a reinforced concrete girder to resist a flexure, including an allowance for its own weight of 2,000,000 in.-lbs. The economical steel ratio is assumed to be 0.008, other assumptions as stated above.

Solution.—In the table the K is stated as 78. Then

$$b\,h^2 = \frac{2{,}000{,}000}{78} = 25{,}640 \text{ ins.}$$

If b be assumed at 15 ins.

$$h = \sqrt{\frac{25{,}640}{15}} = 41.3 \text{ ins.}$$

This proportion of b to h being satisfactory, the steel section required is 0.008 × 15 × 41.3 = 4.96 sq. ins. This suggests the use of five 1-in. rods. They could all be put in one layer. Adding 2 ins. below their level, the working depth of the beam becomes 43.3 or in round numbers 43.5 ins. The required beam would be 15 × 43.5 ins. and would be reinforced with five 1-in. square rods.

The foregoing was the basis of determination of the cross-section of the girder and of the steel in the tension flange. The vertical reinforcement was given no less consideration. The recommendation of the French authority Christophe not to permit beams in which the average unit shear oversteps 14 to 30 lbs. per sq. in. to go without vertical reinforcement of steel, supported as it was by experiments by Professor Johnson, was adopted. Accordingly stirrups or frames made of Ransome rods were used in all girders where the shear exceeded about 20 lbs. per sq. in. These stirrups were made of two and in some cases of three U's, bent and wired together to template. By being thus accurately made they aided in the correct distribution of the lower flange rods. The stirrups were set in planes transverse to the girder and usually vertical and were cross-sectioned and spaced so that at intervals along the beam not exceeding its depth there was a sufficient cross-section of stirrup to take up in tension enough of the shear to leave a balance which, divided by the gross cross-section of the girder, would amount to not over 25 lbs. per sq. in. In some of the smaller girders and at critical points in the largest ones the stirrups could take up the whole shear without overstepping 16,000 lbs. per sq. in. in the steel. Care was taken in all cases to see that the stirrups had sufficient anchorage above and below the middle of the girder to develop their tensile strength without pulling through the concrete. As a further precaution in this line, the rods at the open ends of the stirrups were bent at right angles.

The construction of the girders is illustrated by the drawings of Fig. 2 showing the plain girders carried by the columns of Row C, Fig. 1, and by the drawings of Figs. 3 and 4, showing the upper and lower girders, respectively, carried by the columns in Row D, Fig. 1. The drawings are so plain that little explanation is necessary except in connection with the cantilever beams. The purpose of this construction was to retain the same cross-section throughout the curve as was used on the tangents and to some extent to counterbalance the rotary effect of the curvature of the girders. The ends of the suspended beam and of the cantilevers being critical points subject to very severe shear they were stepped so as to reduce the effective concrete area as little as possible, and were reinforced with especial care. The stirrups used here were of special construction; were placed in an inclined position and were designed to resist the whole vertical shear.

The cantilever ends tend to act as joints at which the shrinkage stresses are relieved. The first suspended span to be built shortened in hardening and slipped on the treads of one of the slipped ends, so as to show a crack through the extent of the risers at the joint, and in slipping spalled the corners of the step. After this four 1-in. rods were put in at mid-height of the girder and lengthwise with it, crossing the joint and extending into both the cantilever and the suspended beam far enough to develop the strength of the rods, with a view to preventing this slipping. So far as observed the result has been satisfactory. Shrinkage joints are thus kept about 115 ft apart in this line. At the ends of these intervals are opportunities for the shrinkage to take place harmlessly. Besides using rods, the steps were finished off with a troweled surface

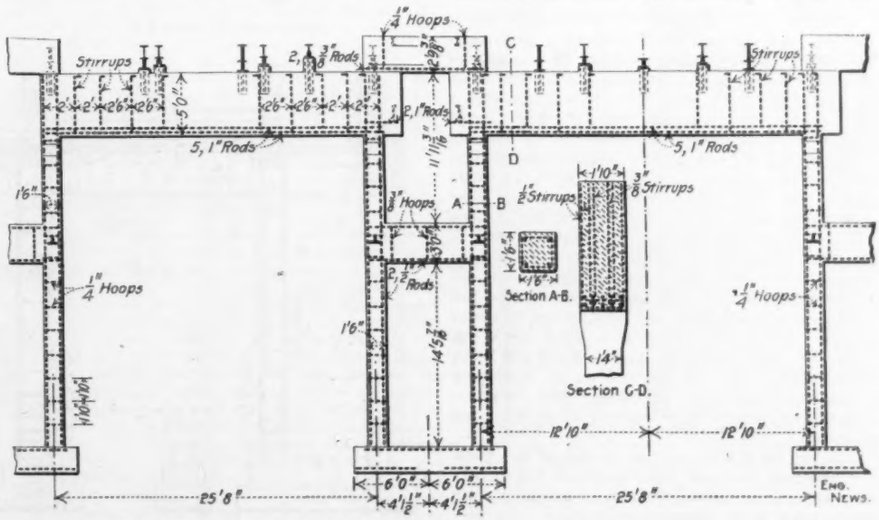



FIG. 2. DETAILS OF COLUMNS AND GIRDERS IN ROW C, ON CURVED PORTION OF HARVARD STADIUM.

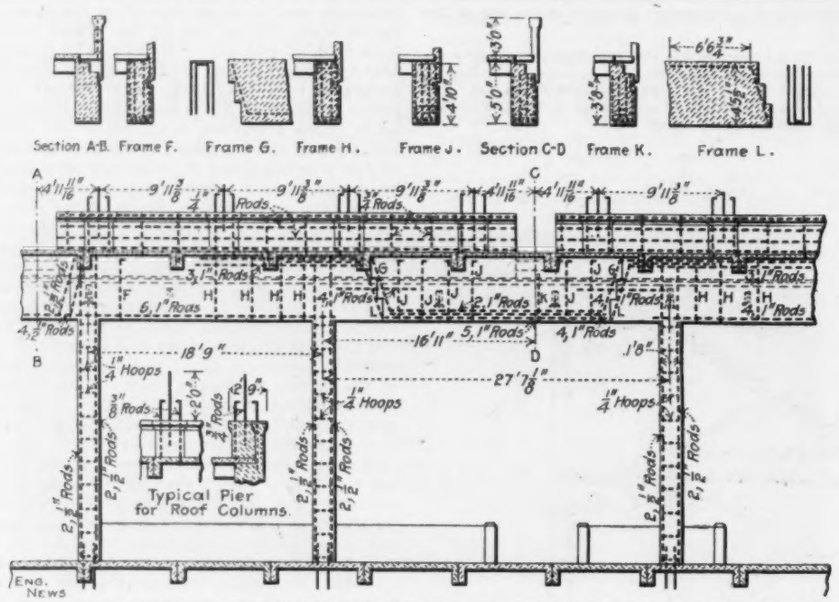



FIG. 3. DETAILS OF COLUMNS AND GIRDERS IN ROW D, ABOVE MEZZANINE FLOOR, HARVARD STADIUM.

truly level, so as to have things in shape for a harmless slip should the rods prove ineffective.

FLOORS.—The promenade floors were made of ribbed slabs about 8 ft. 3 ins. wide and 18 ft. 6 ins. in span, cast alternately in place, thus providing shrinkage joints at the edge of every slab. The thickness of the body of the slab is 4½ ins., exclusive of granolithic finish, and the flanges are 6 × 18 ins., making practically a 4½-in. flat floor resting on 12 × 18-in. joists spaced 8 ft. 3 ins. on centers, and having a span of 16 ft. in the clear.

SEAT SLABS.—The seat slabs are a series of L's set up so as to form a flight of treads and risers. They are of crusher dust concrete poured at the consistency of cream and reinforced as shown by Fig. 5. The slabs on the semicircle were made curved; all the others were straight. As indicated by Fig. 1 the slabs are supported by brackets riveted to the inclined I-beams carried by the inside wall and the three succeeding rows of columns.

WALLS AND PARAPETS.—The ends of the U forming the stadium are closed with plain curtain walls between solid columns. This construction calls for no particular description. The same is true of the inside wall, Row A, which is also a solid curtain wall between columns. The outside wall is of rather special construction. It consists of a series of wide piers in two stories carrying girders and surmounted by a parapet wall, the whole giving the effect of arcades. The actual construction is indicated by the section, Fig. 1, and by the special elevations and sections of Fig. 6. The wall piers are designed in the same general way as were the solid columns previously described, with four vertical rods at the corners, but they were made hollow, and 66 × 36 ins. in cross-section. The shells on the 66-in. sides are 4 ins. thick and those on the 36-in. sides are 6 and 8 ins. thick. The latter are counted on as furnishing the whole compressive strength. The construction of the girders is sufficiently clear from the drawings.

The special problem in the design of the walls and parapets was how best to provide for shrinkage, so as to minimize the evil of cracking from this cause or from temperature changes. One way was frankly to leave joints at short intervals and free to open; the other way was to attempt to prevent all cracks by the aid of proper reinforcement.

In the outside wall a combination of the two methods was adopted. Expansion joints were inserted at intervals of 16½ ft., but they had to be placed over the centers of the piers, in spite of its being realized that the friction from the weight of the superimposed mass would probably prevent all sliding and thus prevent such joints from being effective. Consequently the wall was heavily reinforced by ½-in. bars, spaced 6 ins. apart, with a view to defying temperature changes. The inside wall was not specially reinforced in this way, but expansion joints were inserted at 16½ ft. intervals. These joints opened perceptibly immediately upon hardening and now constitute open joints sometimes 1-16-in. in width, but changing as the temperature rises and falls. The end walls, which are some 70 ft. long and from 9 to 50 ft. high were reinforced with ¼-in. rods spaced 6 ins. apart. They are without shrinkage joints.

METHODS OF CONSTRUCTION.

Haste was required in the construction of the stadium owing to the need of it for coming athletic contests. This made it necessary to install a rather elaborate plant, the general arrangement of which is shown by Fig. 7. As will be seen the plant consists of two parts, one for the concrete work on the stadium proper and the other for the work of molding the seat slabs.

Considering the stadium work first it will be observed that concrete from the mixer was taken by cars to two cableways. These had inclined carrying ropes and straddled the two arms of the structure. The task of making and depositing the concrete and of placing the reinforcement presented comprised few feature of especial interest. The concrete was deposited in a semi-liquid state and tamped enough to remove air bubbles. The forms were of wood planed and oiled where the surfaces were exposed to view and rough dressed in all other places. The reinforcing rods were twisted at the mill but were bent and cut in the field, a force of six or eight men being required for the work.

In some respects the most interesting feature of the construction was the molding of the floor slabs. There were 4,800 of these slabs and they were cast in sand molds constructed as shown by Fig. 8. The patterns were placed in the flask as shown, and the sand packed around them with the box B inserted to save sand and lighten the mold. The cover A was clamped in place and the flush inverted by a derrick. Cover C was then removed and the pattern withdrawn. The reinforcement previously wired together was then put in place guided by a template. The concrete was composed of cement and crusher dust and was mixed to the consistency of cream which enabled it to be poured into the molds. A gang of experienced molders could turn out about fifteen molds per day for straight slabs and from four or five per day for curved slabs. In warm weather the cast slabs were removed from the molds in two or three days and kept under wet burlap until needed. In setting the slabs in place it was found necessary to do considerable chipping although an allowance of ⅛-in. had been made for joints.

Five months and two days from the setting of the first battle boards and less than five months from the turning of the first shovelful of earth, the stadium was occupied comfortably to its full final capacity.

The engineering design of the stadium was in the hands of Prof. L. J. Johnson, of Harvard University, with Mr. J. R. Worcester, M. Am. Soc. C. E. of Boston, as consulting engineer. Mr. C. F. McKim and Mr. G. B. Gersdorff, both of New York, handled the architectural side of the problem, and the Aberthaw Construction Co., of Boston, Mass., was the contractor.

Acknowledgment is due to the Journal of the Association of Engineering Societies and the Harvard Engineering Journal, both for June, 1914, for the bulk of the material in the preceding article.

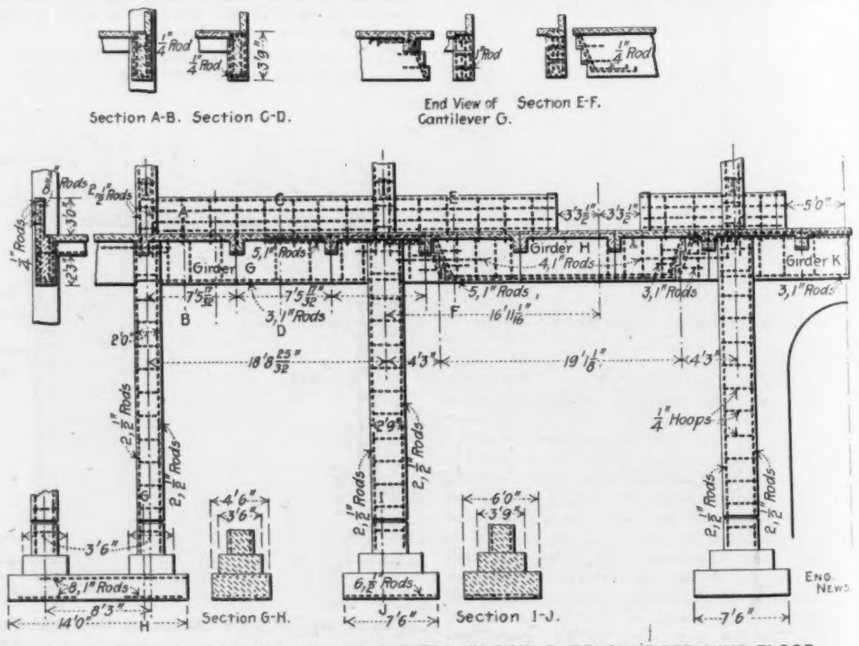



FIG. 4. DETAILS OF COLUMNS AND GIRDERS IN ROW D, BELOW MEZZANINE FLOOR, HARVARD STADIUM.

## METHODS OF EXPANSION OF RAILWAY BRIDGES.*

By Francis S. Rice.†

The question of expansion in railway bridges resolves itself into a practical one rather than one of theory. On the other hand it is treated in the specifications on the part of railways and generally in the practical designing of bridges almost wholly from a theoretical standpoint.

When the necessity of making provision for expansion was first recognized it was naturally treated as now by providing the ordinary type of one plane surface sliding upon another. This simple design is used generally for girders, and for plate girders of moderate span. Most

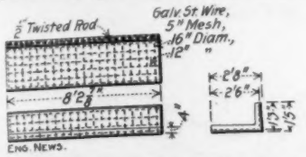



Fig. 5. Details of Standard Seat Slab for Harvard Stadium.

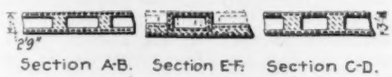
of the railways confine this type of expansion bearing to spans of not over 75 ft.

As spans became longer, heavier weights had to be provided for, and the next step was the introduction of rollers between two planed plates. Both of these types were modified and improved by the addition of a tongue and groove, thus providing against transverse motion without causing friction or binding on the anchor bolts.

The roller type was provided with side bars acting as spacers, and later much ingenuity has been shown in the endeavor to make them proof against the admission of snow, dirt and water. When much heavier spans were designed, it was considered necessary to have larger rollers, and there was objection to their weight and size. This objection was met by the design of the segmental form of roller, which was first used by German engineers so far as we have records.

This covers nearly the whole practice of ordinary expansion bearings in railway bridges. These types are often modified more or less. For instance, some of the later railroad specifications require that dust guards shall be placed on the four sides of roller bearings, and shall be held in place with tap bolts so that they can be removed and the rollers cleaned and oiled.

The best known determination of the coefficient of expansion in steel for one degree of Fahrenheit, expressed in terms of the length, is that ascertained by the United States Government through a series of careful experiments at Watertown Arsenal about 1887. Expressed decimally it is 0.000,006,466 or 1 ÷ 154,655, for a variation in temperature of 150° F. this amounts to 1⅛ ins. in



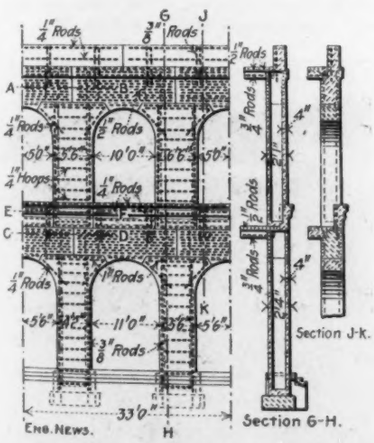




Fig. 6. Details of Outside Wall Piers and Girders for Harvard Stadium.

100 ft. Railway specifications generally call for provision for expansion of 1 in. in 100 ft., which is very nearly this amount. Taking the modulus of elasticity of steel at 29,000,000 lbs., which is the commonly accepted coefficient, and calculating the amount of actual distortion in the steel occasioned by this 150° of temperature variation, we find that the amount of stress in steel, required to produce an equal distortion, would be 28,130 lbs. per sq. in. Stated in a different way, the amount of elongation or contraction occasioned in the lower chord or any other member of a steel structure through a variation of 150 degrees F. is the same as that occasioned in the same member by a strain of 28,130 lbs. per sq. in., and this is very near the elastic limit of ordinary structural steel.

Now as regards certain factors of friction. Roller friction is a very uncertain quantity, and depends so entirely on the finish of the surface of both the beds and the rollers themselves that it is difficult to give any average statement for it. A fair average coefficient would be 0.002 of the weight, which is the stress required to move a body under ordinary rolling friction with such conditions as originally exist in a roller shoe. The sliding friction such as would be occasioned in the first type of shoe, ranges from .07 to 0.11* of the weight for the start, and after the start is made it comes down to as low as 0.004. In other words this sliding friction after the shoe is started is just about twice as much as the rolling friction, if each were in perfect condition.

In regard to the effect of the pressure per square inch on sliding friction, I might say that up to a reasonable limit the pressure per square inch has no effect upon the percentage of weight which would overcome that friction. This reasonable limit is about 500 to 600 lbs. per sq. in. Prof. Thurston is authority for the statement that in the pivots of some center bearing draw bridges the amount of this sliding friction is as high as 9,000 lbs. per sq. in. That seems to me a very high unit stress and I cannot think that it has been intentionally employed in designing. However, the New York Central R. R. Co., in its specifications specifies that bearings on phosphor-bronze can be employed at a unit stress of 5,000 lbs. per sq. in.

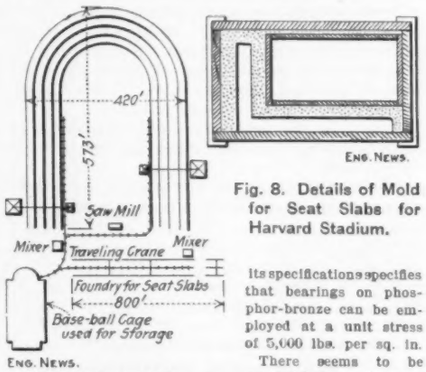



Fig. 7. General Plan of Construction Plant for Harvard Stadium.




Fig. 8. Details of Mold for Seat Slabs for Harvard Stadium.

There seems to be some difference between the theoretical and practical standpoints of this expansion question. Theoretically the best results should be obtained from rolling shoes. Yet I have seen numbers of good engineers, practical men who are acquainted with these things, give a smile, and sometimes a broad one, when the subject of rollers and roller shoes was mentioned. It seems to be a kind of joke among them, that these roller shoes are very nice to put in the drawing, but that after they have been in the bridge five or six months they are rusted and the bridge never moves. The consequences might not be serious in an ordinary span; but for a long span the effect would be worse. I have tried to approximate the effect by the statement that a stress of 28,130 lbs. per sq. in. is caused by a change of 150° Fahrenheit. That seems to be a point where theory and practice conflict, for otherwise since it is well known that the shoes of many bridges do not move, from the above considerations they should pull over their piers at least twice a year.

Just to draw out a little discussion I may say that it is my opinion that we are wrong in adopting the roller shoe. I think we had better go back to the plane or sliding shoe. The difference in the amount of resistance or friction between the sliding and the roller shoe is very small, and is not to be compared with the benefit gained from the eventual maintenance of the feature of expansion in all bridges. I take that ground from the fact that the roller shoe, while efficient when first placed, is impossible to maintain practically. The shoe is below the railroad track in all cases. It is subject to the varying eddies and currents of wind and storm, and catches dust from every passing train; sand and ballast gather around the rollers, the accumulation draws and retains moisture, rust follows, and the shoe is stuck fast, and little is or can be done to prevent that condition. I hold that the difficulties of preventing that condition are so serious that we had better go back to the old plane sliding shoe. It would be easy to design a plane sliding shoe for any span which might have a phosphor-bronze bearing for one of the surfaces, and which would be so tight that if properly lubricated it would be effectually sealed and could be practically maintained in a working condtion.

### DISCUSSION.

**Willis Whited.**—I like Mr. Rice's idea of flat bearings, but almost any kind of bronze would do as well as phosphor-bronze. Instead of oiling it, give both surfaces a coating of a mixture of plumbago and tallow with just enough oil added to make a paste; it will not dry, decay or evaporate, and it reduces friction as much as can be done practically by any other means and requires no subsequent attention.

It is a fact that draw bridges to-day often have 5,000 to 8,000 lbs. pressure per sq. in. on their bearings and they will stand it all right if the bearing is well fitted and lubricated with something like plumbago that will not squeeze out easily.

G. H. Danforth.—The firm I was with in 1896 put in a draw bridge on the New York & Long Branch R. R. It was a center-bearing bridge having two phosphor-bronze disks, with a steel plate between. These disks were figured for about 5,000 lbs. per sq. in., and I think got fully that load. On account of faulty design, it could not be lubricated very well, as the oil could run out as fast as it was poured in.

The bridge was put in in the spring before navigation opened and during the summer the railroad had trouble with it. In the fall I was one of two men to go down there and with the assistance of a section crew, jack the bridge up on a Sunday for an examination. The disks were found in good condition, in spite of the loading, and of the fact that the lubrication had been defective.

T. J. Wilkerson.—I have been advised by reliable authority that on the Memphis bridge Mr. Geo. S. Morison used segmental rollers and placed on one set of rollers a vernier to enable him to ascertain the elongation produced by a heavy train passing over the bridge. When the load passed over, the vernier did not show any elongation; there was no doubt some elongation and it was doubtless taken up by the piers springing.

Referring to Mr. Rice's unit strains of 28,130 lbs. per sq. in. caused by a temperature of 150°: if we assume that the change of temperature is 100°, which is sufficient for most localities, we will get a unit strain of about 18,500 lbs. per sq. in., which added to 12,500 lbs. per sq. in., the unit strain usually allowed for railroad bridges, we get 31,000 lbs. per sq. in., which is less than the elastic limit of good medium steel, hence we would be unable to tell if our bridge has been subjected to higher unit strains than it was designed to carry.

From all evidences I have been able to obtain, it appears that very few rollers ever move after they have been in place sufficiently long to rust.

Mr. F. S. Rice.—Mr. Wilkerson's remarks about Mr. Morison's bridge at Memphis calls to my mind a further case of Mr. Morison's practice. Mr. Morison designed a bridge for the O. R. & N. R. R. over the Columbia river at Portland, and it has quite a long draw span, probably 250 ft., with several approach spans of equal length. The foundations of the draw spans and the approach spans are iron tubular piers filled with concrete which are something like 12 ft. in diameter set on piles extending down into the alluvial bottom a considerable distance. Mr. Morison put in these spans, approaches and draw span without any provision for expansion whatever. Any serious expansion there would seriously interfere with the working of the draw span. I made careful inquiry of the bridge tender and was informed that they had never had any serious trouble with the draw span on that bridge, although any serious expansion in the structure would certainly pinch the draw span and make it hard to operate.

*A paper read before the Structural Section of the Engineer's Society of Western Pennsylvania, Jan. 24, 1905.

†Engineer, Railway Bridge Department, Riter-Conley Mfg. Co., Pittsburg, Pa.

*See editorial comment in this issue.

## THE WATER SUPPLY OF AMSTERDAM, HOLLAND.

By W. P. Mason.*

That Amsterdam, like several other cities of Holland, derives its water from the "Sand-Dunes" is a fact pretty widely known, but it is possible that a knowledge of just how the supply is secured may not be quite so general.

The "Dunes" are irregular round-topped sand hills, which have been erected by wave and wind action along the whole of the Netherlands coast. In height they at times reach 160 ft., but average much less, while the breadth of the range is very variable. Near the sea they are devoid of vegetation, the surface of the sand being moved to a considerable degree by the action of the wind, while further inland grass appears and crops of various kinds are raised.

The particular section, near Haarlem, whence Amsterdam draws its water, is composed of rather loose sand in which the "reed-grass," planted in tufts by the city water department, keeps a very precarious footing. The rain falling thereupon each year amounts to about 28 ins.

The natural "run-off" of the "Dunes" is nearly zero, and the absorption by vegetation still less; 64% of the annual rainfall sinks into the sand.

It is interesting to note that according to the opinion of some writers the water-bearing zone of the "Dunes" is fed in part by the condensation of the aqueous vapor contained in the air which circulates through the spaces between the sand-grains, and also in part by ground water which flows from the interior of the country towards the

*Professor of Chemistry, Rensselaer Polytechnic Institute, Troy, N. Y.

sea and which rises in the sands of the "Dunes" partly by capillarity and partly because of the pressure of the specifically heavier sea water below.*

One theory advanced to account for the ground water of the Netherlands coast, and of the islands adjacent thereto, is that such water might be considered "fossil" in character, it having saturated the strata previous to the sinking of that section of the continent. Possibly it may at that remote

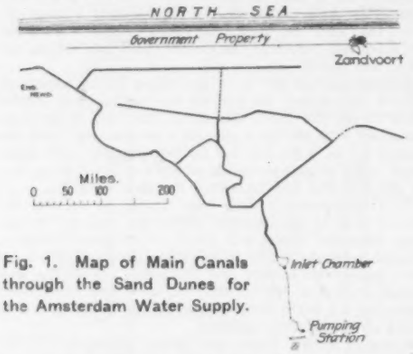



Fig. 1. Map of Main Canals through the Sand Dunes for the Amsterdam Water Supply.

time have been in the state of ice, and have sunk as such with the sinking strata.

According to the views of Mr. Andrimont, the greater part of these waters come by subterranean flow from the interior of the country. This proposition is combatted by Mr. Pennink, who shows by careful measurements that the water of the "Dunes" has its origin in the local rainfall.†

Entirely in accordance with the view that local rainfall alone is responsible for the "Dune" water is the fact that upon sandy islets of the sea at long distances from shore ordinary domestic wells furnish supplies of very acceptable character.

The area drained by the collecting system of Amsterdam is about 7,500 acres and the volume of water secured amounts to some eight million gallons daily. Open canals, very similar to small railway cuts, wind between the sand hills and gather the ground water of the district into a collecting basin, whence it is pumped to Amsterdam, some 14 miles away.

The side slopes of these collecting canals are 1½ : 1, and although the cross sections are not all of the same dimensions, a good idea of their general size may be obtained from the section shown herewith. The depth of water carried in them is now about six feet. The aggregate length of the canals is about 16 miles.

The general level of the local water table has been lowered from 12 to 15 ft. since 1853 by the

Fig. 3. One of the Larger Collecting Canals.

operation of the Amsterdam water system; the case being similar to what has been observed in the chalk wells of London, namely, more water has been removed than could be replaced by the annual rainfall. Now, however, the delivery is fixed at a figure not to exceed the rainfall, and consequently the level of the water table is held constant.

*La Technologie Sanitaire, Dec. 15, 1904.
†Paper by Mr. Pennink before the Royal Institute of Engineers, November, 1903.

The "Dune" supply allows about 50 liters per capita per day (13.29 gallons) to the inhabitants of Amsterdam; an amount which is recognized as insufficient, and steps are being taken to increase the quantity.

Considerable iron is present in the water, but it has given no trouble, as it quickly oxidizes to insoluble oxide which is removed in the ordinary course of filtration. It is worthy of note that an attempt was made to extend the collecting system by means of iron pipes laid with open joints, a plan which is in successful operation under similar gathering-grounds near The Hague; but crenothrix immediately appeared and gave so much difficulty that the underground drains were abandoned and replaced by the older form of open cuts. Why crenothrix is not causing annoyance at The Hague it is not easy to understand, the conditions governing the two collecting areas being so simjlar.

Before delivery to the consumer, the water is filtered through English sand-beds of the ordinary type. They are uncovered and have vertical walls of brick with asphalt bottoms. Each filter is 30 × 59½ meters in area and the underdrains are of D-shaped tile pipe, the lines of which are placed 5 ft. apart. The depth of sand is 32 ins.

Originally the works were controlled by a private company, but they were purchased by the city in 1896.

The raw water of the "Dunes," in the Amsterdam district, contains, in parts per million:

| | |
|---|---|
| Iron (Fe) | 96.8 |
| Chlorine | 32.3 |
| Silica ($SiO_2$) | 7.6 |
| Free ammonia | .19 |
| Albuminoid ammonia | .106 |
| Nitrogen as nitrites | 0. |
| Nitrogen as nitrates | .33 |
| Total solids (at 180° C.) | 344. |

## BRITISH REFUSE DESTRUCTORS.

(Editorial Observations.—Concluded from p. 408, April 20, 1905.)

### Installations Old and New, Including the Largest Destructor Plant in the World.

The notes here brought to a conclusion illustrate the varied character of the destructors visited. As previously explained, most of the plants were visited at random, thus including some of the poorest as well as the best exampics.

### REFUSE DESTRUCTOR AND ELECTRICITY WORKS AT ACCRINGTON.

Accrington has a population of about 43,000 and an area of 3,426 acres. Mr. W. J. Newton, Assoc. M. Inst. C. E., is surveyor, and Mr. H. Gray, M. Inst. E. E., is electrical engineer to the borough.

Six top-fed Horsfall cells, placed in a row, were installed in 1900. They have a total grate area of 180 sq. ft. and a rated daily capacity of 60 long tons. The amount of refuse actually burned, I was told, is from 50 to 60 long tons a day. Back of the cells are 7½ × 30-ft. Lancashire boilers, with a heating surface of about 1,000 sq. ft. each. As these supply steam to an electric light and power plant, supplementary coal-fired grates were placed between the cells and the boilers for emergency use. The electric plant is also provided with an 8 × 28-ft. coal-fired boiler. All the boilers are equipped with Green's economizers. Steam jet blowers are used to produce a forced draft. The chimney of the combined plant is over 240 ft. in height and is of brick. Between the economizers and the chimney is a dust-catcher, which utilizes centrifugal action to deposit the dust in chambers.

The destructor building is of brick. The tipping floor has a stone pavement supported on iron girders. The clinkering floor is of concrete.

The clinkers are raked into small wheeled trucks, run out on a track and dumped into a pit, from which they are fed into a crusher. An elevator takes the crushed clinker to a screen which rejects the dust and all pieces above 2 ins. in size. The clinker is then used for concrete street foundations. Some has been used for filter beds at the local sewage works and a little has been sold to another municipal sewage work.

For the three months prior to my visit the destructor had been run night and day. Previous to that it had been shut down between midnight and 6 a. m. On the 24-hour basis the works are operated in three shifts, with one top man, one stoker and one wheeler-out to each shift. The men are paid from 6½ to 5 d., or 13 to 10 cts. per hour.

The refuse which I saw on April 15, 1904, consisted almost wholly of ashes and contained no green stuff whatever.

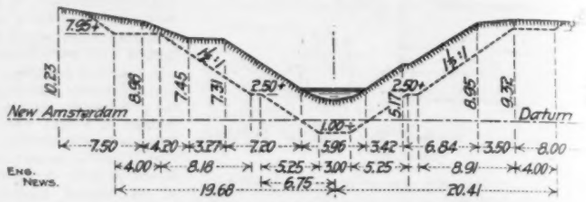



FIG. 2. TYPICAL CROSS-SECTION OF CANAL.

The electric equipment includes five engines with a total of 970 HP. and five dynamos with a combined capacity of 580 KW. Chloride batteries of 750 ampere hours' capacity are also provided.* There were only 300 electric consumers at the time of my visit. The output last year was about 280,000 Board of Trade Units.

Mr. Gray estimates that a fifth of the steam consumption is supplied by the destructor boilers. It has been found from tests that a unit on the consumer's meters costs about 0.4 d., or 0.8 ct., when obtained from coal, and on figuring the yearly averages of cost anything under that figure is credited to the destructor. Last year 0.2 d. was so credited (which does not seem to agree with the foregoing statement that the destructor furnished one-fifth of the power required, unless there has been a large recent increase in the load).

Mr. Gray himself thinks, and believes all electrical engineers agree, that it is not best to combine refuse destructors and electricity works. He much prefers coal for fuel, on account of the excessive labor cost for handling refuse.

The actual quantity of refuse burned in November, 1901, was 2,132,256 lbs. The destructor was shut down Sundays, so the average quantity burned per day was about 82,000 lbs., or about 36.5 long tons.

A 22-hour official test of the destructor and one boiler was made on April 11 and 12, 1901. A total

Fig. 4. New Filter Beds Under Construction at Amsterdam.

of 117,846 lbs. of unscreened mixed refuse was burned. This was equivalent to 21,424 lbs. per cell per 24 hours, or 29.7 lbs. per sq. ft. of grate surface per hour. A total of 135,624 lbs. of water was evaporated, which was at the rate of 1.15 lbs. of water per lb. of refuse at actual tem-

*The figures in this sentence and the results of tests given below, are from Goodrich's "Refuse Destructors for Power Production" (London, 1904).

perature and 1.39 lbs. calculated from and at 212° F. The mean steam pressure was 185 lbs. The mean temperatures were as follows: Main flue, 1,000° F.; behind the boiler, 500° F.; feed water, 60° F. The clinkers and ashes amounted to 35.5% of the weight of the refuse burned. Mr. Goodrich (see foot note) gives the labor cost at this plant as about 33 cts. per long ton and the electric output at 25 Board of Trade Units per long ton of refuse burned.

The total cost of the destructor, buildings and chimney, exclusive of site, was about £8,000, or a little less than $40,000.

## YORK.

I had time for only a hasty glance at one of the two destructors at York and found that new electric works are receiving all steam not needed for operating the destructor accessories.

York has a population of about 78,000 and an area of 3,692 acres. Mr. Alfred Creer, Assoc. M. Inst. C. E., is city engineer.

The plant which I saw consists of four Fryer's top-fed cells, erected in 1898, equipped with Boulnois & Brodie's charging trucks. A fan is used to provide forced draft. The buildings were of the solid construction common to destructors in Great Britain.

The clinker is crushed and variously used, some going to a mortar mill at the works. Tins have recently advanced in price from 3½ to 5 shillings, or from 85 cts. to $1.22 per long ton.

## AN OLD DESTRUCTOR AT BOURNEMOUTH.

Among the older destructors in England and one of the few that still rely upon natural draft, is the one at Bournemouth. This popular seaside health and pleasure resort has a population of about 60,000, distributed over an area of 5,850 acres. Mr. F. W. Lacey, M. Inst. C. E., is borough surveyor of Bournemouth.

The destructor now includes six top-fed cells, of which four were built by Manlove, Alliott & Fryer in 1887 and two by Goddard, Massey & Warner in 1891. The differences between the two installations are chiefly in minor details. The chimney is of brick and has a height of 137 ft.

Coal has never been used to aid combustion, and although the destructor, like all of an early date, is of the low temperature type, provision was made at the outset for utilizing some of the otherwise waste heat. To that end the heat was used in preparing material for tar concrete sidewalks. The plant was given up on account of the long haul for both the raw and prepared material. At present the heat is by-passed, when desired, beneath iron plates on which sand is spread to dry for use on rails of the municipal tramway system, a branch of which passes near by.

A Jones fume cremator was in use for about three years, but it was expensive in operation, and as there were no complaints against the destructor when the cremator was not used, it was shut down.

As is the case at nearly every refuse destructor which I saw abroad, the ramp or inclined roadway here is of earth. The tipping space is paved with stone.

All the refuse is collected by the borough. The collecting equipment includes two large steam dust vans, or motor wagons, made by the Thorneycroft Steam Wagon Co., of Basingstroke. The vans hold four or five ordinary wagon loads. They are provided with steel boxes and each has a screw, work by a crank, for dumping.

The refuse brought to the works is not weighed. It is estimated to aggregate 40 long tons a day, which appears to be about half the total amount collected by the borough. The balance is tipped.

A recent proposal to provide a second destructor was thwarted, owing to the fact that adjacent property-owners, who had a veto power, objected to the site chosen. The present site is near the center of one side of the borough and distant from annexed territory on either hand. There are a number of small houses within 400 to 500 ft.

Some of the clinker is used for road bottoming, but none is sold. The tins are dumped in deep fills in new roads and compressed by a steam road roller.

The destructor is run night and day, except that the fires are banked and the dampers closed from Sunday morning to early on Monday morning. Six men are employed, including the foreman, in two shifts of three each. The men are paid 29 shillings, or $7.05, a week, and the foreman receives 30 shillings, or $7.30, a week. This makes a payroll of $42.55 a week, or about 17 cts. per long ton for 240 tons of refuse per week.

I visited the destructor on April 7, 1904. In the several loads of refuse which I saw there was scarcely any green stuff, and the foreman said that most of such as did come was from markets. A slight odor was noticed when I first came alongside the cells; evidently from the charging floor. The chimney gave out but little smoke.

The foreman at the destructor has worked there ever since it was opened, and has been in the employ of the municipality for 37 years.

## REFUSE DISPOSAL AT GLASGOW.

Glasgow has five destructor installations with an aggregate of 59 cells, but not all of its refuse is burned. The bulk of what we should call garbage is shipped by rail to one of several farms owned by the city.

The population of Glasgow is about 780,000, and its area is 12,688 acres. Mr. A. B. M'Donald is city engineer and surveyor, and Mr. D. McColl is superintendent of the cleansing department.

Three of the older destructors, with a total of 30 cells, were designed by the department. Of these two have chimneys 250 ft., and one has a chimney 300 ft. high. A plant of five Horsfall top-fed cells, erected in 1898, has a chimney 250 ft. high, steam jet blowers, and a Babcock & Wilcox boiler which supplies steam for the blowers and for works purposes.

THE DALMARNOCK DESTRUCTOR.—This plant is located at the Dalmarnock sewage works. It was erected in 1901 and in 1903, and includes 16 grates, provided with steam jet-blowers for forced drafts. The chimney is 250 ft. high. The cells are of the Meldrum top-fed type. Three Lancashire boilers supply steam for the blowers, for lighting the works and for breaking and screening clinkers. The surplus heat is delivered to the sewage works.

The clinker is sold for use in making concrete. The solder is recovered from the tins, and the scrap iron, bottles and the like are sold.

THE MARY HILL OR RUCHILL DESTRUCTOR.—The eight Fryer's top-fed cells of this plant were installed in 1902 by Manlove, Alliott & Co., of Nottingham. Two 8 × 30-ft. Lancashire boilers supply steam to drive fans for forced draft, a clinker crusher and screen and a dynamo to light the works. The chimney is 200 ft. high. Golfers on hilly ground near by complain of the chimney, but, I was told, without cause.

The rated capacity of the destructor is 80 long tons a day, and from 60 to 70 tons are now brought to it. Everything that comes is burned, unless it has a marketable value. I saw hundreds of old shoes that had been put aside for sale.

The destructor is operated in two eleven-hour shifts of five men each, not including the crusher men. One shift extends from 6 a. m. to 5 p. m., and the other from 6 p. m. to 5 a. m.

I visited this destructor on April 18, 1904. Nearly all the refuse which I saw was ashes. No odor was noticed and the chimney was giving out but little smoke.

## BRADFORD.

Bradford has four destructors with an aggregate of 38 cells. Each plant has a chimney 180 ft. high and uses steam jet blowers to produce forced draft. At three of the destructors the steam raised is used for works purposes, only, but from one of them steam is delivered to electricity works.

The population of Bradford is about 280,000 on an area of 22,843 acres. Mr. Ernest Coll is cleansing superintendent. I made hurried visits to two of the destructors on April 21 in company with Mr. John Watson, Managing Director of the Horsfall Destructor Co., of Leeds.

THE SUNBRIDGE ROAD DESTRUCTOR.—Nine old cells of another make have recently been rebuilt and three cells added, all of the Horsfall top-fed type, with storage at the rear. Each cell has a grate of 30 sq. ft. area. The charging holes are stopped by the refuse, instead of by lids, and the refuse is pushed onto the fires through these holes, instead of being hauled forward or pushed in from the rear through doors reached from the lower floor. A hood is placed above the clinkering doors and from this warmed air is drawn to be forced through the grates at the rear of the cells and down into the ashpits.

The cells are placed in a single row and behind them are two Babcock & Wilcox marine type boilers, rated at 200 HP. each. The rated capacity of the destructor is ten long tons per cell per day or 120 tons in all, and the contract guarantee for rebuilding called for a yearly electrical output of 1,000,000 Board of Trade units. There is a Green's economizer and electric storage batteries will be added.

The destructor building and the chimney are of brick, and the ramp, tipping and clinkering floors and clinker yard are paved with stone. A dust catcher, based on centrifugal action, is provided.

The clinker is raked into buckets working on overhead trolleys, one to each cell. There are two mortar mills for utilizing the clinker and a dynamo for lighting the works.*

When I visited the destructor scarcely any green stuff was visible in the refuse, and but little smoke was seen at the chimney top.

The standard wage for laborers at all the Bradford destructors, Mr. Watson said, is five shillings, or $1.22, per eight-hour shift, during which each man handles an average of seven long tons of refuse; that is, one charger handles 14 tons and one stoker clinkers the same amount.

SOUTHFIELD LANE DESTRUCTOR.—There are six cells here, similiar to those just described. There is also one 8 × 30-ft. Lancashire boiler, two clinker crushers, an elevator, a rotary screen and a mortar mill. The crusher is simply two fluted chilled iron or steel rolls.

The ramp is stone-paved, with iron wheel tracks. There are a mess-room and a bath-room for the workmen.

TESTS OF THE HAMMERTON ST. DESTRUCTOR.†—The first destructor at Bradford was erected at Hammerton St. in 1880 and 1882. About 1890 forced draft was added, and in 1897 the twelve cells were rebuilt by the Horsfall Company. The plant is equipped with two 8 × 11-ft. multitubular boilers which supply steam for jet blowers, a flag- or slab-making plant and other works purposes.

A test extending over 278 hours was made June 24 to July 7, 1900, during which 2,896,320 lbs., or 1,293 long tons of mixed refuse were burned. This was at the rate of 9.3 long tons per cell per 24 hours or 34 lbs. per sq. ft. of grate surface. A total of 2,153,000 lbs. of water was evaporated, or 0.743 lbs. actual and 0.882 lbs. from and at 212° F. per lb. of refuse. The temperatures were as follows: Gases in main flue, 1,800° F.; gases at chimney base, 1,000° F.; feet water, 60° F. For each ton of refuse burned there were 83.2 I.H.P. The residuals amounted to 379.65 long tons, or 29.36% of the original refuse, divided as follows: Clinker, 364.96 tons; fine ash, 12.02 tons; fine dust, 2.67 long tons.

The average weights per cubic foot of different classes of refuse consumed during the test were as follows: Ashpit, 42.2 lbs.; market, 22.6 lbs.; light refuse, 19.2 lbs.

The labor charges during the test were: 6 chargers at 25 shillings or $6.05 a week and 12 stokers at 28 shillings or $6.80 a week, making an average of 9 d., or about 18 cts., per long ton.

## THE LARGEST REFUSE DESTRUCTOR IN THE WORLD: HAMBURG, GERMANY.

The lagest refuse destructor in the world, if I mistake not, is located at Hamburg, Germany. Other cities have a greater aggregate of cells, but not under one roof. The Hamburg destructor, however, serves only a portion of the city. The refuse from other sections is used for agricultural purposes or is tipped, as was all the refuse of the

*In the "Public Health Engineer" (London) for August 20, 1904, it was stated that a rental of £120, or $583, a year was derived from power supplied from this destructor to run a weaving shed; also that the municipal corporation has decided to fully utilize the heat from this destructor by supplying steam to a new electric station, not far distant, for lighting and street car purposes.

†Prepared from figures given in Goodrich's "Refuse Destructors and Power Production" (London, 1904).

city before the destructor was installed. I am indebted to Mr. Rud. Schroder, Chief Inspector of the pumping and filtration division of the Hamburg water-works, for taking me to the destructor, introducing me to Mr. Otto Uhde, Chief Engineer and Manager of the plant, and serving as an interpreter.

The destructor was built in 1895 and put in operation in January, 1896. Until January, 1901, it consumed the refuse from a population of about 300,000, and since that time the tributary district has been increased to include a population of 420,000, or perhaps a half of the whole city.

The plant comprises 36 top-fed Horsfall cells, in units of six cells, placed back to back. Originally the forced draft was provided by means of steam jets, but soon after the plant was accepted the city substituted fan blowers. The contract guarantee was for 5 long tons of refuse per cell, but on changing the nature of the forced draft this quantity was exceeded and now from 7 to 10 long tons per cell are burned. Doubtless other factors, such as extra labor per unit for stoking, have contributed to the increase. The chimney is of brick, approximately 8 ft. in internal diameter at the base and 160 ft. high.

I saw monthly diagrams showing the operations of the destructor for years. Less refuse per cell is burned in winter than in summer. Last summer the maximum rate was 10 long tons per cell, as compared with 8 and even 7 tons in winter. Mr. Uhde stated that this difference was due to the fact that the greater proportion of waste coal in the winter refuse seemed to result in a coking process. The summer temperatures in the destructor are also higher than those obtained in the winter. Among the many refuse destructors and garbage furnaces which I have visited in Europe and America this is the only one at which I ever saw a pyrometer.

No coal is used to assist combustion. The coal used throughout the city, I was told, gives a residue more favorable to refuse destructors than that burned in Berlin, where brown coal is used and the cinders have little fuel value. This statement is of interest in view of the alleged failure of some experimental destructor cells tested at Berlin a few years ago. The refuse which I saw at Hamburg on May 13, 1904, was largely composed of ashes, with some paper and old tins, but scarcely any green stuff.

Refuse is collected at night in wagons having removable boxes of 4 cu. m. or 5.2 cu. yd. capacity. There are four reversed hooks on each box, to which four chains, hanging from an electric overhead traveling crane, are attached. The boxes are then lifted, moved laterally, and tipped onto a storage platform adjacent to the charging holes. There is one feed hole to each pair of cells. The tins are not picked out of the refuse.

The cells, or most of them, are equipped with hand-worked movable grate bars. The sides of each cell, for about 8 ins. above the grates, are cooled by means of water.

Four multitubular boilers are provided for raising steam from the gases of combustion. Each boiler has a heating surface of 200 sq. meters or 2,152 sq. ft., and in case of emergency each can be fired with coal. Engines of 260 HP., effective, drive a dynamo which furnishes the power required at the plant and supplies current for storage batteries on a steam launch and for operating four 40-E. HP. sewage pumps at a station four miles distant. Another electric pumping station is proposed. The electric launch is used to collect refuse and night-soil from all the boats that come into the harbor. The night-soil is emptied into the sewers and the refuse is brought to the destructor and burned.

Clinkers are removed at intervals of 1½ hours, and are said to equal 50% of the original refuse. The clinkers are raked into small cars, run into a tower in the yard and quenched by means of water jets. An attempt to remove the iron from the clinker by means of a magnet proved unsuccessful and the iron is now picked out by hand. The clinker is crushed, after which it is screened to three sizes. Some of the larger material is used for pavement foundations, for concrete, for filter beds at sewage works elsewhere and for sidewalk slabs or flags. The fine stuff is used to fill in beneath floors. The dust from the screen is exhausted through a pipe to a chamber where it is deposited by means of falling water.

Except on two holidays the plant, from year end to year end, is never completely shut down. Some cells are closed on Sundays, but enough are kept going to supply power to the sewage pumping station.

The men work in eight-hour shifts and for each shift there is one charger to each six cells and one stoker to each three cells, making 6 chargers per shift or 18 per 24 hours, and 12 stokers per shift or 36 per 24 hours. The chargers are paid 3.9 marks, or 97.5 cts., and the stokers 4.1 marks, or $1.025, per shift. This is approximately $1 a day for 44 men. Assuming an average of 8 long tons per cell per day gives a total of 288 long tons burned at a cost of $44, or 15 cts., per long ton for charging and stoking.

I was told that a cell equipped for experimental purposes with a Root blower and fixed grate bars, and stoked every ½ instead of 1½ hours, had burned as high as 27 long tons a day.

A mess-room and shower baths are provided for the workmen, and once in about two weeks the workmen's clothes are sent to the municipal disinfecting station.

The chimney is of brick, about 8 ft. in diameter at the base and 160 ft. high. At the time of my visit it was giving out only a light, vapory smoke.

## THE NEW REFUSE DESTRUCTOR AT ZURICH, SWITZERLAND.

One of the most recent destructor installations in Europe and one of the comparatively few on the Continent was opened at Zurich early in 1904. Through the kindness of Professor Carl Hilgard, M. Am. Soc. C. E., of the Zurich Polytechnic School, I was introduced to Herr Gluck, the head of the health department of Zurich, and to Herr Mettler, the inspector of public health. The four named visited the destructor on May 7, and as nearly all the fires were out, we improved the opportunity to inspect the main flue, which is here placed between the double row of cells.

Zurich has a population of about 160,000. Owing to the high cost of coal, the chief fuels used are wood and gas. Most of the garbage is collected by farmers and others and fed to hogs. As a result of these local conditions, the refuse at Zurich is quite unlike that found in Great Britain. The total amount collected by the city is now about 500 tons a week, all of which, including tin cans, is passed through the destructor. Some street sweepings are included in the above total.

The twelve cells are of the Horsfall top-fed type and have a grate area of 30 sq. ft. Forced draft is supplied by a fan with a capacity of 7½ cu. ft. per sec., driven by a 15-HP. electric motor. The chimney is of brick, 6½ ft. in diameter at the base and nearly 200 ft. high.

Two Babcock & Wilcox boilers, with a provision for independent coal-firing, supply steam to a 220-HP. steam turbine. The later drives a dynamo which operates a traveling crane and the fan already mentioned, besides supplying current to light the destructor. Any surplus heat, I was told, will be delivered to the municipal electric light and power plant. The boilers are fed with condensation water.

The refuse wagons arrive at the destructor on the ground level and as old wagons were still in use, the whole of each wagon, instead of the box only, was hoisted by the overhead traveling crane and emptied as best it can be on top of the destructors.

The clinkers are raked into small wheeled trucks and run outside on rails.

The destructor is operated continuously from 5 a. m. Monday to 10 p. m. Saturday. Chargers and stokers alike are paid only 4½ francs, or 90 cts., for an eight-hour day. Their wage should be increased, I was told, to 6 francs a day, since the cost of living in Zurich is high. This is particularly true of rents, three rooms and a kitchen costing 450 to 500 francs a year.

A handsome five-story brick apartment house has been erected by the city, close by the destructor. It contains seven sets of family apartments, besides rooms for single men and an office. It will be heated by steam from the destructor. The apartments and single rooms will be rented to the men employed at the destructor.

## NOTES ON CONCRETE BLOCK MANUFACTURE.

Early last spring a practical builder in New York conceived the idea of making a hollow block machine of wood and, I must say, built one of the best home-made machines I have ever seen, and I have seen many. The first few hundred blocks passed muster, but the wood soon warped and zinc lining was resorted to, but the buckling of those sheets soon made its appearance upon the face of the block and iron castings was the next step taken. Unfortunately, mechanical engineering, much difficulty was experienced in getting a machine that would make a mechanically true block, and as a last resort he purchased a machine made and put on the market by a concern that makes such machinery their business, and now, in summing up the season's work, this builder finds that he would have saved over $400 in labor and material had he purchased a tried machine.

Blocks made on a home-made machine have less air space, hence more material is required and therefore more cost; besides, no home-made machine will admit of the heavy tamping so essential to well made blocks, therefore a home-made machine block is not as strong as a block made on a shop machine. I recently visited a plant where the owner still contends that a home-made machine is just as good as any, and of about 600 blocks I saw in his yard, over 500 were swelled on the sides, which was caused by the springing of the mold while tamping.

The use of lime, hydrated lime and limoid has been fully tried the past season and I fail to find one firm that has profited by it. Blocks made with lime composition are more solid in two to twelve weeks than a cement block, but at six months it is spongy and will soak more water than a block made of cement composition. Let every hollow block maker realize that the life of lime is limited to from seven to sixteen years, and that it is only a matter of time until any block containing lime will crumble.

The chief fault I find with the form of blocks now made is that the openings are in many instances too large and at the same time not sufficient area of the blocks are taken up with air spaces, viz.: My ideal block is one of many small air spaces, at the same time having less material in it than the average hollow block of to-day, and I might add many small binders and spaces make the best hollow block.

Many hollow block makers have gone into the cement brick business and in the next year or two we will see much of this industry's experimental stage. The object to be attained is the production of a brick at less cost than our red brick and at present too much cement is required to make them waterproof to bring their cost below the red brick figure.

Hollow concrete block construction must sooner or later be governed by some form of specifications that will insure sound buildings and not be a hindrance to the builder and a detriment to the trade generally, as is now the case with several building codes that are more theoretical than practical.

Some codes require blocks so constructed that the openings do not exceed one-third of the area of the wall. This is a gross injustice to concrete, and concrete block men should look into this at once in their respective cities, as it has been fully proven that a concrete wall having about 40% of the area hollow makes the best walls, being more damp proof than with less open space.

The height of blocks is also often specified, while there is little or no difference in the strength of a wall built of blocks 4 ins. high than that of a wall of blocks 14 ins. high.

Length of blocks and length of openings have much to do with the strength of the wall and must be limited, the withes (binders) must also be governed as to size and shape. The thickness must be in harmony with the height of the wall, and if this is not properly proportioned damaging results may follow. Some codes compare concrete with brick in proportioning walls, and this is an injustice to concrete, as it excels brick in both tensile and crushing strength. The following outline for a specification for a building code will, I believe, give universal satisfaction, and sufficient buildings have been erected along those lines to warrant its enactment:

### GENERAL CONCRETE SPECIFICATIONS FOR HOLLOW WALLS.

All concrete used in constructing hollow walls, either with hollow blocks or monolithic, must be made of sufficiently sound material and of such proportions to insure a tensile strength of 240 lbs. to the square inch and a crushing strength of 1,000 lbs., said test to be made when concrete is 28 days old.

Openings within said walls not to exceed 48% of the area of wall, unless same be reinforced with an approved system of steel, in which case said openings shall not exceed 60% of said area.

All binders (withes) must not be less than 20% and not more than 45% in volume of hollow spaces. Hollow spaces shall not exceed in length the extreme thickness of the wall.

No block shall be longer than four times the thickness of the wall, and blocks of less length than the thickness of the wall must not exceed 4% of the blocks necessary to construct entire wall.

Many builders wonder why architects generally do not take readily to hollow block construction. The idea that architects are opposed to concrete is wrong, but their

*Extracts from a series of articles by Fred. W. Haglock in "The National Builder" for January, February and March.

...sons for avoiding its use in building construction can be ...tled down in one sentence, viz.: The hollow concrete ... makers in America who produce an artistic block ... can be adapted to harmonize with an architect's de... are few and far between.

... classical architecture Germany has many fine ex...les of artificial stone, and one of the main causes ... we have not the same is that our hollow block ...s have been carried away with the rock face fad, ..., in spite of all efforts in that direction, is to-day but ... imitation of the cheapest possible face that can ...ven to natural stone—a face which excels only as a ...atcher and absorber of water, as it remains an un...ted fact that a well-made artificial stone has that ...ed appearance which kills the naturalness of rock ... work.

...ully nine-tenths of the concrete block buildings I have ...n have been made in imitation of rock face, a finish which I have opposed for the past three years, in spite of the fact that it has been continually becoming more popular, which I attribute to no other reason than that many have entered the field who know absolutely nothing about architecture. The concrete block problem does not, in my opinion, require a superior knowledge of engineering or architecture, but does require both skill and taste, which are two features that are badly lacking with the trade generally, and nearly all the failure in the industry of the past year can truthfully be laid to these deficiencies.

## DREDGES FOR WORKING GOLD-BEARING GRAVELS.

One of the remarkable developments in the production of gold within recent years, is the successful recovery of fine gold from gravel deposits and the introduction of special machinery for the efficient and economical performance of the work. Gold dredging was first practised in New Zealand, but is now in use in this country on a large scale, not only in the dredging of river beds, but also in what may be called land gravel. In this latter case the machine works practically in a pond or pool of water, excavating on one side and filling in on the other, so that the pool moves with the dredge. This method of working is now in use in several States, and especially on very large areas of gold-bearing gravel deposits in California, as described in our issues of July 16 and 30, 1903. In the Oroville district alone, about 6,000 acres of dredging lands are being developed, and 29 dredges are at work. The value of these Oroville properties is estimated at about $45,000,000. While there is usually plenty of water for flotation in these gravels, there are large areas of similar land underlaid by gold-bearing gravel where sufficient water to float a dredge is not available, but no satisfactory means of working these deposits has yet been devised. In the operation of the gold dredges, large tracts of rich fertile lands and fruit orchards have been converted into a desert of rough piles of gravel, and this action had led to an agitation somewhat similar to that which was aroused in the same State years ago by the disastrous results of discharging into the rivers and streams the refuse from hydraulic mining. No definite opposition to the dredging work has yet been made, but it has been suggested that the "slimes" or fine waste material now discharged as thin mud into the water, should be pumped up and discharged on top of the bank of waste gravel, so as to partially fill the interstices. The value of gold product in the California gravels ranges from 10 cts. to 40 cts. per cu. yd.; the latter is very high for a large tract, and 25 cts. per yd, is a very remunerative amount. The cost of operating the dredges is said to be from 5 cts. per cu. yd. handled in the larger machines of about 70,000 yds. monthly capacity, to 6 or 6¼ cts. per cu. yd, in machines of about 40,000 yds. capacity.

In the early experimental stages of the work in this country some dipper dredges were employed, but the violent disturbance of the gravel and the water caused the fine but heavy gold to settle deeper into the ground, while a large proportion of that taken up with the gravel flowed away with the water escaping between the side and bottom of the dipper. The intermittent supply of material also necessitated the use of a hopper or bin to ensure a steady supply to the gold-saving tables, In hydraulic dredging, the force of suction immediately at the mouth of the pipe is sufficient to carry the gold, but at a little distance from the pipe this force diminishes, so that only the sand and gravel are drawn in, while the gold drops to the bottom by its greater weight. In New Zealand, the ladder dredge was used from the first, as this type of dredge is generally adopted abroad, and in discussing this matter in our issue of March 10, 1904, we pointed out that experience has proved the ladder dredge to be the best type for this particular class of work. It causes comparatively little disturbance of the gravel bed, and the material in the buckets cannot escape, while the water brought up in the buckets assists in the washing of the material.

The practical demonstration of the advantages of the ladder dredge have led to its general adoption, with the introduction of various improvements to adapt it for this special work. The machines carry the apparatus for recovering the gold, and while the dredging buckets bring up the gold-bearing gravel at one side of the machine, a conveyor discharges the waste or tailings at the other side. Steam power is used in some cases, but by far the greater number of machines in California are operated by electricity, the current being transmitted from a power station to several dredges. In this way the cost of operation is greatly reduced. Fig. 1 is a sectional elevation of a typical gold dredge, and Fig. 2 is a view of dredge No. 2 of the Boston & California Dredging Co., at Oroville, Cal. Fig. 3 is an interior view showing the gold-saving tables, behind and beyond which may be seen the shaking screens. These machines were built by the Bucyrus Co., of South Milwaukee, Wis., which has made a specialty of dredges for gold placer dredging, and to which we are indebted for plans and information.

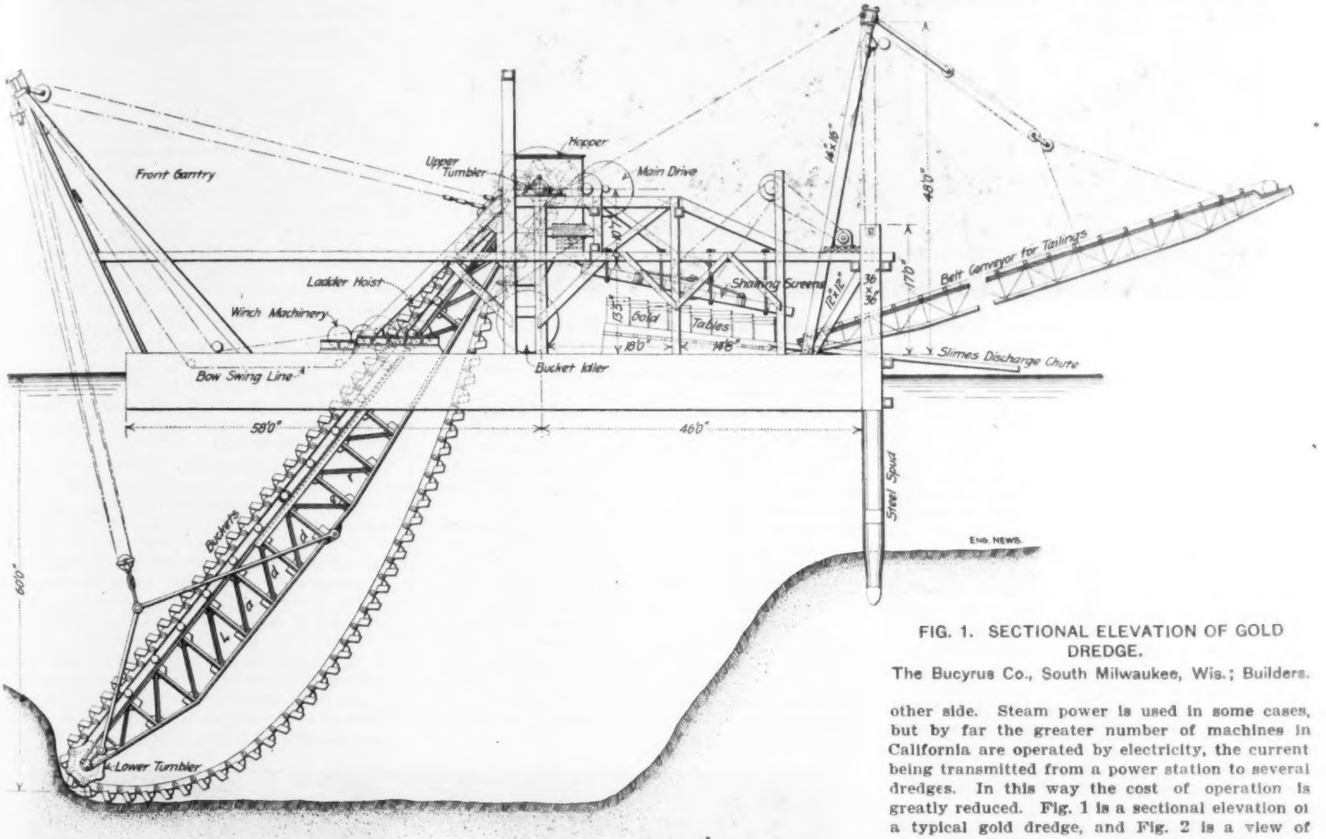



FIG. 1. SECTIONAL ELEVATION OF GOLD DREDGE.
The Bucyrus Co., South Milwaukee, Wis.; Builders.

The dredges have wooden hulls, built on the work, and the head frame from which the lower end of the ladder frame is suspended is usually of wood, although some large dredges have the head frame built up of structural steel. The buckets are of steel, with heavy lips or cutting edges of manganese steel. They are close-connected, each link of the ladder chain carrying a bucket. Where many boulders are liable to be encountered, however, intermediate links are used, so as to allow room for these boulders between the buckets. The buckets are of 3 to 7½ cu. ft. capacity, and travel normally at a speed of about 50 ft. per minute; this may be increased in very light work, or reduced to 30 or 40 ft. in hard ground. In some dredges the gravel is passed through a rotary screen, into which streams of water are delivered, and the fine material containing the gold is passed through a long inclined sluice box having riffles for collecting the gold. This arrangement was shown in our issue of July 16, 1903. A more efficient method, especially when the gold is very fine, is to use shaking screens of rectangular form. From the ends of these screens

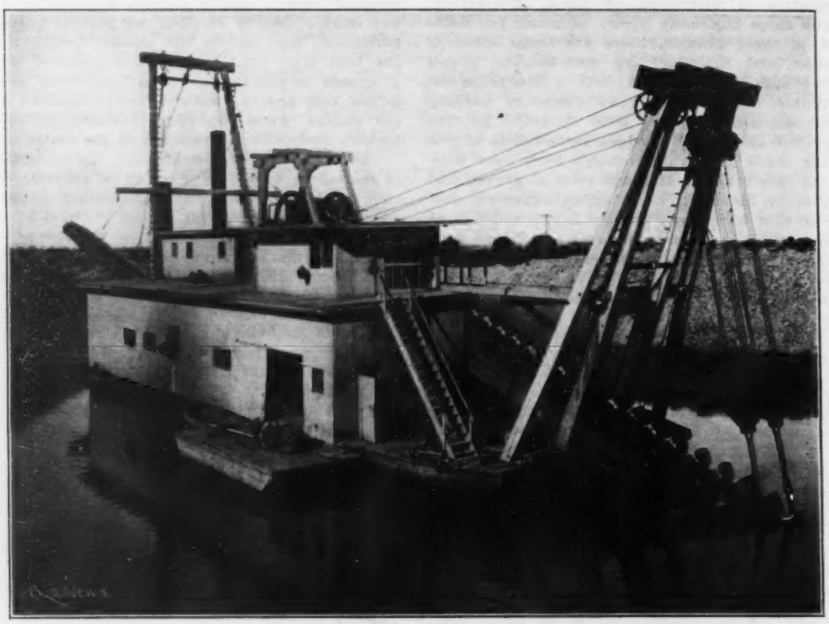


FIG. 2. GOLD DREDGE OF THE BOSTON & CALIFORNIA DREDGING CO., OROVILLE, CAL.
The Bucyrus Co., South Milwaukee, Wis.; Builders.

the coarse gravel passes to the tailings elevator. The fine material and the gold pass through the screens into a distributor which distributes the material uniformly over the gold-saving tables, which are fitted with riffles or other devices according to the character of the gold. The waste from the tables is delivered into sluice boxes, which are also fitted with riffles and extend beyond the hull, delivering the waste "slimes" into the water. When there is a large amount of sand and dirt a centrifugal pump is sometimes employed to handle this waste and deliver it on top of the pile of gravel tailings. When very coarse material is to be dredged, the material is sometimes delivered first to a rotary screen and then to the shaking screens. The waste gravel from the screens is delivered upon an inclined belt conveyor carried by a long frame projecting from the stern of the dredge and supported by cables from a gallows frame. A steel spud at the stern holds the dredge in position, and the bow is swung laterally along the face of the work by means of bow lines operated by winches.

The dredge "Yuba No. 2," working in the Yuba district, is a good example of these machines. Its hull is of timber, 115 × 36 ft., with a draft of 5 ft. The ladder chain carries 88 buckets of 6 cu. ft. capacity, and will dredge to a depth of 60 ft. below the water line, while the top tumbler is 24 ft. above the water line. This machine excavates a maximum of 4,200 cu. yds. of gravel per day of 24 hours, or from 95,000 to 105,000 cu. yds. per month. The tailings conveyor is 116 ft. long. The motor equipment is as follows: 125 HP. motor for driving the ladder chain, 50 HP. for pumps and sand pump (elevating fine tailings), three of 20 HP. each for the screens, tailings conveyor and winch, and one of 7½ HP. for the priming pumps. Alternating current of 440 volts is employed. The shaking screens are 7 ft. and 7 ft. 9 ins. wide, 16 ft. long. The gold tables have an area of about 800 sq. ft., and quicksilver is used to collect the gold. There are two spuds at the stern: a wooden spud for stepping up, and a steel spud to hold the dredge when working.

From Mr. W. P. Hammon, General Manager of the Boston & California Dredging Co. and the Boston & Oroville Mining Co., we have some further details of work in the Oroville district. An elevator or ladder dredge with close-connected buckets of 5 cu. ft. capacity running at a speed of 18 buckets dumped per minute, should handle (in ordinary excavation) an average of 2,000 cu. yds. per day of 20 hours actual running time. This running time is considered a good average, taking into account the shut-downs from all causes. Nine men are considered a full crew, working in three eight-hour shifts of three men each. With a modern dredge, a charge of 5 cts. per cu. yd. should cover the full cost of the gravel excavated, washed and stacked. It is difficult to give an average value of the gold recovered per cu. yd., owing to the varying conditions of the gravel and the gold deposit. In some sections of this district the ground averages 20 cts. per cu. yd., while in other sections it averages 15 cts. There are 29 dredges now working in this district all operated by electricity. The current is obtained from the Valley Counties Power Co., which has several generating stations located on the mountain streams in that section of the country. The current is delivered to the machine at 4,000 volts, and transformed to 440 volts for the motors. As to the supply of water, it is necessary in many cases of operating these inland dredges to have a steady supply of 50 miners' inches of fresh water entering the pits in which the dredges work.

## A MOVING PLATFORM ELEVATOR FOR TEAMS AT CLEVELAND, OHIO.

The use of inclined moving platforms, or "escalators," for carrying passengers from floor to floor of buildings has become quite common in cities, and similar arrangements have often been suggested for handling wagon traffic on steep streets. So far as we know, however, the only escalator for street traffic that has ever been constructed has just been put in operation at Cleveland, Ohio. This device is known as the Snead Rolling Road, and is in operation on Factory St., where it overcomes a rise of about 65 ft. in a length of 420 ft. In the accompanying views, Fig. 1 shows the completed road in operation, and Figs. 2, 3 and 4 show portions of the partly completed structure and aid in explaining its construction.

Briefly described, the "rolling road" consists of an endless belt traveling around two sets of sheaves, one at the top and one at the bottom of the hill, and supported between sheaves by an upper and lower truck. The belt is composed of planks laid transversely of the road and fastened together in "trucks" of two planks each by steel plates, so connected as to form flexible chains. It is given motion by driving sheaves at the top and the bottom of the hill, which are operated by electric motors. One of these driving sheaves is shown by the view Fig. 2, which also shows a portion of the top roller truck on which the belt rides. This top roller truck is carried by steel T-beams which span the pit in which the belt operates, as shown by Fig. 3. Fig. 4 shows a portion of the lower roller truck on which the belt returns; this truck is carried by concrete buttresses built into the side walls of the pit. A stationary platform projecting over the pit and belt at each end permits of driving wagons onto and off the moving platform or belt.

Details of the construction of the moving platform are withheld by the inventor, but the above description gives a general idea of their character. The platform weighs about 106 tons and is operated by four 25-HP. Westinghouse motors controlled by a Westinghouse R 7 controller placed in an operator's house at the top of the hill. In operation the platform is stopped and the team driven onto it at the lower end; the platform is then started and the team is conveyed to the top of the hill when the platform is again stopped to permit the team to be driven off. It is possible, of course, by stopping and advancing the platform as often as teams approach the lower end to convey a procession of teams at the same time, as is

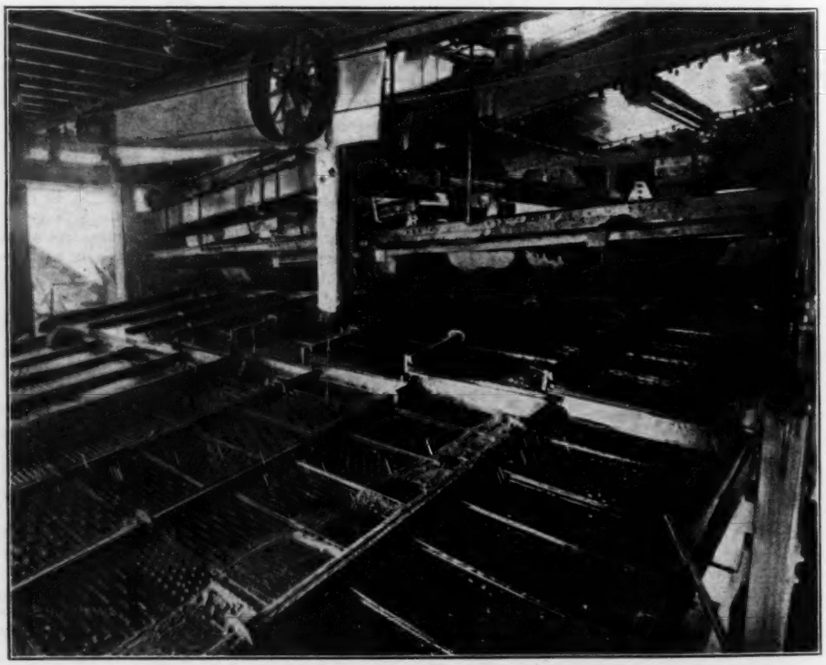


FIG. 3. INTERIOR VIEW OF GOLD DREDGE, SHOWING THE SHAKING SCREENS AND GOLD-SAVING TABLES.

FIG. 1. MOVING PLATFORM ELEVATOR FOR TEAMS ON FACTORY ST., CLEVELAND, OHIO.

indicated by the view, Fig. 1. The roadway is calculated to carry a load of 100 tons and to operate at a maximum speed of 3 miles per hour. At this speed the belt makes a half revolution in 2 minutes and 10 seconds, and when only two stops are made a team can be taken from the top to the bottom of the hill in 4 minutes.

The Factory St. rolling road was built by the Cleveland Rolling Road Co., after designs by Mr. Isaac D. Snead, Civil Engineer, of Cincinnati, Ohio. The company charges tolls ranging from 50 cts. for a heavily loaded four-horse team to 15 cts. for a single horse, wagon and driver. Pedestrians and passengers other than the drivers are charged a toll of 2 cts. The road has been in operation since January of this year. It cost about $100,000 to build.

## METHODS AND COST OF CREOSOTING TIMBER.*

By Oswald T. Dunn, C. E.

Creosoting as a preservative against decay is almost universally used, and has long been proved to be a method which protects those parts injected with it absolutely.

*Condensed from an article in "The Polytechnic," March 31, 1905.

The creosote (dead oil of coal tar) used in this process consists of that portion of coal tar which distills between 200° and 300° C; together with the residual oils from the manufacture of crude carbolic acid, naphthalene and anthracene. It is greenish yellow. The smell is unpleasant and highly characteristic. It is heavier than water, its specific gravity being as high as 1.10.

At present it is believed that the preservative action of the creosote oils is due more to the bases, or alkaloids, than to the tar-acids, as the former remain after the latter have disappeared.

The cylinders of to-day are made of ⅝ or ¾-in. iron and vary from 6 ft. in diameter and 100 ft. long, to 9½ ft. in diameter and 150 ft. long. They are provided at one or both ends with doors having a gasket to make a tight joint, and means of fastening to the end of the cylinder to prevent the breaking of the joint under the internal pressure. The cylinders are fastened at the middle to prevent any displacement and permit movement towards the ends, due to large changes in temperature.

The timber is drawn into the cylinders on narrow gage tram-cars which provide for the handling in and out of the cylinders quickly and cheaply.

After the timber has been drawn into the cylinder, and the doors securely fastened, live steam is admitted until a pressure of from 10 to 40 lbs., depending upon the size of timber, is reached and prolonged sufficiently to allow the heat and inward force of the steam to gradually raise the inside temperature of the timber above the living point of the bacteria, thus accomplishing sterilization, and at the same time causing the evaporation of the sap and moisture. During the steaming period, the cylinder is frequently drained from a valve, or steam trap, at the lowest point, in order to keep it as free as possible from sap and condensed steam. Great care is taken to avoid overheating the timber, it being now determined that temperature over 300° F. has a very damaging effect upon the strength of timber; which is due to resinous substances evaporating at about 305° F., thus destroying the structure of the wood. Wood dried at 300° F. is composed of over 99% of organic and less than 1% inorganic matter, and at this temperature wood gives off only water, though some of the slight chemical changes are noticeable, and if the heat is increased, gases of pungent odor and taste are evolved. Thus the vulcanizing process is claimed to effect a chemical change in the constituents of the sap itself, being practically a seasoning process.

The steaming is regulated, generally, according to the following schedule: For material with a minimum dimension of from 1 to 4 ins., 30 lbs. For material with a minimum dimension of from 5 to 8 ins., 35 lbs. For material above 8 ins., 40 lbs. For piles not less than 7 ins. in diameter at the small end, 40 lbs.

At the end of the steaming period a vacuum is created in the cylinder, and the temperature maintained by means of steam passing through heating coils, all the time above the boiling point corresponding to the existing vacuum. After steaming for a specified length of time, the timber is saturated, to some degree, with water which is highly charged with sap and other elements dissolved out of the wood, and it is desirable to withdraw this water and also the air in the wood cells, to make room for the preservation solution. If the temperature in the cylinder is maintained above the vaporizing point corresponding to the degree of the vacuum, the water will leave the timber and pass over to the vacuum pump as vapor, and will not depend simply upon the expansion of the air behind it to force it out.

After the sterilizing and drying of the timber, the solution is admitted, flowing through a 12-in. pipe from elevated tanks of several thousand gallons capacity.

The oil enters the cylinder in the usual fluid state at not less than 120° F., and should be raised to not less than 180° F. by means of the heating coils in the cylinder. When the cylinder is filled with the liquid, force pumps are employed to force the solution into the cells of the wood under a pressure of 100 to 150 lbs.; and the pressure is kept up till the system of measurements show that the required amount of oil has been absorbed by the timber. The pressure is then taken off and the timber removed from the cylinder.

The general method of filling the cylinder is from the elevated tank and is accomplished in about seven minutes. Some fill by means of the vacuum pump, in which case the solution is drawn into the cylinder by the vacuum, and of course, requires a longer time to fill.

The gaging of the quantity of the solution put in a cylinder charge is done by means of a float on the inside of the storage tank attached to a gage graduated to feet and decimals of a foot.

The only objection against seasoning before treatment is the time required, but experience has taught that in the long run the saving from better service far exceeds the cost.

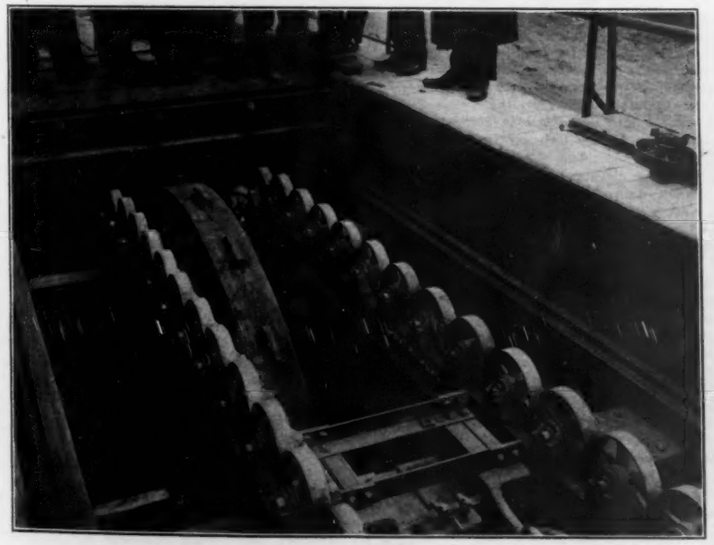


FIG. 2. VIEW SHOWING DRIVING SHEAVE AND UPPER ROLLER TRUCK FOR MOVING PLATFORM.

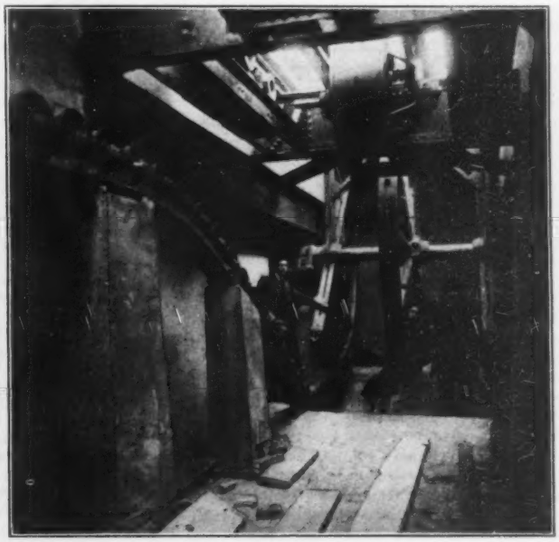


FIG. 4. VIEW SHOWING DRIVING SHEAVE AND LOWER ROLLER TRUCK FOR MOVING PLATFORM.

The steaming of wood before the injection of the solution can never replace seasoning as a means of preparation for treatment, for at best it just drives off part of the water.

The class of timber, whether heart or sap wood, must also be considered. The more sap there is in timber the longer it will last when treated, as the sap-wood is more easily injected and absorbs more of the solution because it allows a greater water penetration than the heart wood.

The time required to complete the different processes varies from 6 to 36 hours and while there is no definite data which will admit of a fair comparison of time required to treat seasoned and unseasoned timber, I give an example that came under my own personal notice. A charge of pine piles just from a river, were steamed for 10 hours; received a vacuum of 26 ins. for 6 hours; and then required a pressure of 150 lbs. continued for 4½ hours to force 2,700 gallons of creosote into the charge. The second charge of pine piles had been seasoned, to my knowledge, one year. They were steamed for 3 hours; received a vacuum of 26 ins. for 1½ hours, and required 30 mins. to force 2,700 gallons, at a pressure of 100 lbs. into the charge.

The cost of treating consists of the interest on capital, solution, labor and fuel.

Creosoting is the most expensive process, costing from $15 to $20 per M., while the burnetizing is the cheapest, costing from $2 to $5 per M.

Assuming that two 6-ft. cylinders 100 ft. long are used, the capacity of each will be about 16,800 ft. B. M. The total plant required would cost, say, $80,000. If the timbers were to be creosoted and impregnated with 20 lbs. per cu. ft. it would take about 36 hours for a run, and the capacity of the plant per annum would be 6,088,800 ft. B. M. If the interest on capital (including repairs and renewals) is 10%, the interest on plant will be $8,000, or $1.14 per M.

If the oil costs 8 cts. per gal., the cost per M. is $15.33. The cost of labor and fuel is assumed at $3.75 per M., which makes a total of $20.22 per M. The cost of green timber, transportation and profit would be considered if the work were taken by contract.

The Eastern Railway of France has obtained the best results of any road in Europe. They use beech timber for ties and inject 12 lbs. per cu. ft. These ties have lasted 30 years. Experience in this country has shown that creosoting is the only reliable method for preserving timber which is exposed to salt water, and the Teredo worm.

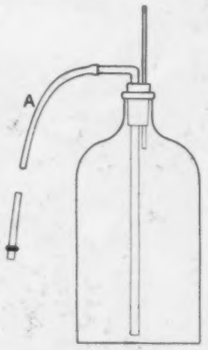
Treated with 20 lbs. per cu. ft., piles in the bridge of the Louisville & Nashville Railroad over the mouths of the Pascagoula River have been in the structure 28 years and will be in use for many years to come. These piles are subject to attacks of the Teredo Navalis at a point where uncreosoted piles a foot and a half in diameter have been cut off by the Teredo in a single year.

## A DEVICE FOR COLLECTING SAMPLES OF WATER FROM VARIOUS DEPTHS.

By W. L. Butcher,* Assoc. M. Am. Soc. C. E.

Sanitary engineers, especially those engaged on water supply investigations, often find it desirable to collect samples of water from various depths in ponds or reservoirs.

The usual method of performing this operation is to lower a bottle enclosed in some sort of harness, and weighted so as to cause it to sink. After reaching the required depth, the stopper is removed by means of a cord reaching to the surface. This method of obtaining the desired end is at best a long and cumbersome process. Extreme care must be used to prevent the two cords, one holding the bottle, and the other attached to the stopper, from becoming entangled while in the case of large samples, the large weight which would be necessary to sink the bottle makes it necessary to collect the sample in portions.



A Device for Collecting Water from Various Depths.

The device shown in the accompanying sketch, although an extremely simple one, has not, so far as the writer is aware, been used before for the purpose mentioned. It consists simply of a stopper, through which two glass tubes are passed. The short tube acts as an air vent, while the long one, reaching nearly to the bottom of the bottle, has a rubber tube attached to its outer end. This rubber tube, carrying a small weight at its open end, is lowered to the required depth. On immersing the bottle and tube below the surface water flows in from the desired level. Water which has obtained access to the tube while being lowered can, of course, be run into the bottle, and then removed before collecting the final sample. If the rubber tube is pinched tightly at the point A while its end is being lowered, hardly any water will enter the open end. After the syphon has started, the bottle may be lifted slightly, if desirable, so that the hand may be clear of the water during the operation of filling.

*211 Lake View Ave., Cambridge, Mass.

Fig. 3. View Showing Pit and Track Girders for Moving Platform.

## ESTABLISHING GRADES AND MONUMENTS IN SMALL TOWNS.*

By Ralph B. Slippy.†

Ackley is a town of about 1,700 population, in Hardin County, Iowa, at the junction of the Illinois Central and Iowa Central railroads. In the spring of 1904 a special election was held to determine the advisability of having a complete resurvey of the town to establish both the property lines and the grades. The proposition: "Shall the survey be made and the grade established," was carried by a small majority, which would have been larger but for the predictions of the opponents of the measure, who claimed that the work would cost the town from $1,500 to $2,000, in addition to an untold amount in defending law suits. On May 24, in response to a request to come and make an estimate of the cost, the writer visited the town and entered into a contract to perform the work at a stipulated rate for his own services, the town to pay all other expenses. The question as to what monuments to use was discussed at considerable length. The town council desired that the grades be marked in some permanent manner, as the expense of getting an engineer would be almost prohibitive on account of the very little work that would be needed in the future. The writer proposed a cement block monument, though the danger of their being moved by frost and by interested parties was fully presented. The cost of the whole work of making the survey, establishing the grade and of setting a cement block monument at each block corner was estimated at $650, which estimate was not far from the actual cost. The monuments themselves cost 14 cts. each or a total of $70 for the 500 required. They were made by a local cement worker in a wooden box form in the shape of the frustum of a pyramid with a square base, the top being 3 ins. square and the base 6 ins. square. They were 24 ins. long and weighed about 40 lbs. each. A 40d. spike was placed in the top of each block while the cement was green, which served to indicate the precise line of the survey.

The survey of the town was undertaken first. [illegible] the government corners and the few monuments re[illegible] on the original plat (which bore the date of 185[illegible] had been found, the center line of each street was run [illegible] temporary stakes were set at the exact intersections [illegible] center lines of the cross streets. The temporary [illegible] were made from hard wood lath of the kind u[illegible] building fences and were cut in lengths of about [illegible] and were tack pointed. From these intersection a[illegible] temporary stakes of the same kind were set at [illegible] block corner. The transit was set over the inter[illegible] peg and line taken to fixed sights on each street and [illegible] set 35 ft. each way by measure and carefully lined [illegible] ft. being one-half the width of the streets. From [illegible] pins the block corners were set by measure and the [illegible] porary stakes set with tack points. The angles bet[illegible] the street lines were read and recorded as a check [illegible] work.

Following the survey, the preliminary levels were run and the grade system worked out. On the grade map there given the grade for the crown of the street, the curb grade for each curb corner within the limits of the intersections and the walk grade for each corner when this was not set to the walk grade. On account of the great number of figures necessarily appearing on the grade map the surface elevations were not given, these being shown on the profiles. The profiles were made for the streets in both directions and showed the grade line and surface profiles of the center line of the street in black ink, the profiles of the side lines of the street in colored ink, and gave also the elevation of the cement walks already in.

Having the grades thus worked out in detail the next step was to distribute and set the grade monuments. The temporary stake was first "pegged in" and then the hole was dug with a post auger or spade to the proper depth as determined by the level and rod. The top was placed so that the spike head was in the correct position as regards line and grade and the earth was then tamped around the block. It was found that the best time was made with three men to dig the holes and set the monuments, 40 blocks being an average number set in a day with this force. The work of getting them to the exact level was not nearly so great as expected, as after setting a few the men could place them with about the same number of trials that a stake could be driven to grade, although the work of lifting them was considerably heavier, on account of the weight of the blocks. It was found that where the blocks would have to extend more than about one-third of their length above the ground, it was better to simply set them in and record their elevation. A number had to be reset because of their being too easily moved when held only by a mound built around them.

The city council provided an ordinance requiring all improvements to be made in the future with reference to this survey and made it a misdemeanor to disturb the monuments except to use them legitimately. The grade ordinance itself established the grades in one direction only but made the grades of the cross streets to correspond with the grades of the streets named in the ordinance as shown by the grade map and profiles on file with the city clerk, thus saving a considerable item of expense in advertising.

This is a somewhat detailed description of a system that will meet with the approval of the town councils of many of the small towns. It will undoubtedly find objection among many engineers in that it delegates the work of setting walk grades to those not competent to do so. At the same time it must be admitted that any one who could not set a walk grade when he has a monument at each end of the block and has the elevation given on the grade map with the proper grade for the corner, would not be able to use engineer's stakes if he had them. Any good cement man can get a very good grade and line without the use of any instrument other than pointers.

As regards the design of a grade system, it is a question passed over by a great many engineers without much attention to very important details. Grades are often established which do not provide sufficiently for drainage, a most important consideration in the writer's belief. The minimum grade should be not less than 1 ft. in 400, although we sometimes must be satisfied with less. It is no serious mistake to place one side of the street higher than the other, but the difference should not be more than 2 ft. between the curbs. A greater difference can be obtained by terracing between the walk and curb. If this is done, however, care should be taken to prevent too steep an approach from the cross streets and alleys. A width of from 30 to 36 ft. between curb lines is ample for most small towns, though the tendency is to make this greater. Deep side ditches are unnecessary. In one town in which I established grades in the past season, the town council wanted me to lay out a system of ditches to cover the whole town and it was only after considerable argument that they were convinced that if the streets were properly graded the water would find its way out. This one occurrence, strengthened by ideas equally absurd which I am continually meeting, convinces me that there is much room for education before we as engineers find municipal engineering in small towns entirely satisfactory.

*Abstract from the Proceedings of the Iowa Engineering Society, January, 1905.

†Waterloo, Iowa.

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Why should New York City be prohibited from generating electricity at the great dam or dams which it is proposed to build for its additional water supply? There is no justification for such a prohibition on the part of the legislature, and the only explanation which we have heard or can imagine for barring the city from utilizing this valuable source of power is that private light and power companies object to having the city as a possible competitor. A similar utilization of power was included in the plans for the Wachusett dam recently completed by the Metropolitan Water and Sewerage Board near Clinton, Mass. The U. S. Reclamation Service, in building irrigation works in the West, will install power plants to utilize the fall of water between the large storage dams and irrigation canals. Nearer home hydro-electric power is being developed at the lower end of the Chicago drainage canal, where the canal discharges into the river. This electric current will be transmitted to Chicago for the use or benefit of that city. As pointed out by Mr. Charles D. Flinn, Assoc. M. Am. Soc. C. E., in Engineering News of Sept. 13, 1900, in an article on the Wachusett dam, a large deep storage reservoir, supplying a conduit heading far below the normal water level of the reservoir, affords an unusually good opportunity for developing water power, since both the head and the supply of water are far more constant than in the case of water supplies from ordinary stream flows or from small mill ponds. The magnitude of the additional water supply for New York, it may be added, will afford an unusual opportunity for this class of hydro-electric development, and one that might be made to yield or save the city millions of dollars within the next 25 years alone. To compel the city to allow this power to go to waste would be an injustice; to shut the city from its use and subsequently allow owners of private capital, whether the present lighting monopoly or other favored persons, to gobble up these rich privileges, with no or at best inadequate compensation, would be a political crime. Mayor McClellan has perceived this and his appeal to the public to arouse itself in self-protection should be heeded. His action in this and other matters pertaining to the much-needed water supply for New York is far-seeing, public-spirited and deserves the commendation and support of the public.

The net result of the investigation of the recent disastrous boiler explosion at Brockton, Mass., is that the old-fashioned cylindrical boiler with lap-seams contains a certain element of danger, that no more such boilers should be built, and that as fast as possible existing boilers of this type should be displaced by boilers of safer design. The monthly report of the Boston Manufacturers' Mutual Fire Insurance Co., contains the following statement by Vice-President Lane:

> This explosion furnishes little new information by which such occurrences may be prevented in the future wherever lap-seam boilers are still in use. This boiler, a 72-in. horizontal tubular, built of ⅝-in. plate with lap-seams, is one on which we customarily recommend a pressure of about 78 lbs. when new, based on the so-called factor of safety of 5. Many insurance authorities, however, allow and even recommend a factor of safety of 4½ or even 4, and in special cases we should not consider the pressure of 90 lbs., which was being carried, as excessive.
>
> So far as can be learned from the investigation of our inspectors and published evidence, the cause of the explosion was the cracking of the plate under the lap-seam —a defect practically impossible to determine by inspections of any kind. It is the same defect which has caused nearly all the bad explosions in this type of boiler. It can be obviated only by the use of "butt-seam" boilers and by discarding wholly the lap-seam boilers.

It is probable that boiler insurance companies will in future refuse to insure lap-seam boilers except when they are of recent construction, of good workmanship and well-cared for.

Many large railway repair shops in this country are poorly equipped for economical or rapid output, especially when they are overcrowded with work and have to handle locomotives much larger and heavier than those for which their machinery was originally designed. These shops have, in fact, not been improved in regard to equipment to enable them to meet the constant demand for greater output and greater economy of work. Within the past few years, several railways have effected radical improvements in this respect, but very much yet remains to be done. The improvements may be broadly divided into two classes: First, the establishment of entirely new shops, in which is sometimes concentrated the work of two or more old plants; second, the reconstruction and enlargement of old shops, to increase their capacity and efficiency. In these improvements one of the most important features is the machinery and machine tool equipment, and here again the methods practised may be broadly divided into two classes: First, the introduction of new machines and tools on a large scale; second, the overhauling and improvement of existing tools to increase their capacity and efficiency.

The importance of this subject, and the general interest which railway mechanical officers take in it, was very clearly shown at the March meeting of the Western Railway Club, when a paper was presented by Mr. M. K. Barnum, who had charge of the design of the new Moline shops of the Chicago, Rock Island & Pacific Ry. (described in our issue of Feb. 11, 1904), and who is now engaged in expert work for the mechanical department of the Chicago, Burlington & Quincy Ry. The paper stated briefly the uneconomical and unsatisfactory conditions which now exist to a large extent owing to the retention of old shop tools and machines in spite of the introduction of more and heavier locomotives, and the heavier work required on the locomotives. The main point made in the paper was that a railway should establish a depreciation fund to provide for the renewal of its shop machinery in order to maintain its efficiency in a proper ratio to the work required of it.

While the paper was short, it provoked a long and energetic discussion, which almost universally supported Mr. Barnum's views, but also showed very clearly the trying conditions under which many railway repair shops actually work owing to financial limitations which preclude the possibility of installing new equipment, even in the face of greatly increased demands upon the shop. The value and economy of a systematic renewal and improvement of shop equipment was acknowledged, but in the absence of such a system, the mechanical department must find some way of doing more work with the old equipment.

Among other matters, the discussion developed the opposite views of those who believe in the introduction of new tools and those who believe in improving old tools and methods to meet new conditions. On the one hand, the new tools may be idle a large part of the time owing to the amount of work being insufficient to keep them at work. But, on the other hand, the improvement and strengthening of the old tools may be carried to a very uneconomical extreme. We have already shown that the purchase of new tools is sometimes impossible. But even where the mechanical department has a free hand, no hard and fast rule can be drawn as to whether to buy new or improve old equipment. The best method in any one case must depend upon the character of the shop equipment, the character of the locomotives which have to be dealt with, and the character of the work required. In many cases, both methods can be followed simultaneously, some parts of the old equipment being replaced and others improved.

It appears that a very far-reaching revision and improvement of shop methods and machinery has taken place on the entire system of the Atchison, Topeka & Santa Fe Ry., and special means have been taken to determine not only the time required for the separate operations of a piece of work, but also the cost of these operations, with the proper allowance for depreciation, power, supplies, etc. To such an extent has this detail investigation been carried, that every belt is numbered and a record is kept of every belt failure, and the life and performance of every belt. The objection was made that all this must involve so much clerical work and so much labor in looking after the belts as to far exceed any economies. The reply was, however, that it takes only a portion of one man's time to keep the record, that it does not require a large clerical force, and that even counting in the cost of the extra men, the practice has proved to be a very profitable investment.

While the introduction of better tools and methods is very important, the human element must be considered, and is, perhaps, equally important, for unless the shop management has the support and active cooperation of the men the desired results cannot be attained. A few instances will show the great importance of this consideration of the human element. In one case mentioned the output of a boring mill that for years had been turning out 20 wheels a day, was increased to 80 wheels a day by the introduction of a premium system. With the use of this same system throughout the shops, the number of locomotives handled was increased by from 57% to 70% each month, with a reduction of 14 to 26% in the cost for each engine. In another case, a general overhauling of equipment and methods resulted in reducing the time for engine repairs from 60 days to 22 days for the same work. Such results, however, can only be obtained with a loyal and experienced shop force; but many railways cut down the shop appropriations when business decreases, so that it is necessary to discharge men or reduce their time. The result is that good men will not stay, and when there comes a demand for increased work, the force has to be recruited with such men as can be had. Even if these are good workmen, they are unfamiliar with the work and methods of that particular work and shop, and the results are not favorable. It was also pointed out that some large shops designed specially to give economy and efficiency of work have been established in the open country, at inconvenient distances from towns. The discomforts and inconveniences resulting from this are such that it is difficult to attract and keep good men. The men are not mere machines to be worked for a certain number of hours, but they require (and rightly so) some consideration for their leisure time and their living accommodations. In regard to this phase of the subject, one speaker remarked that the limit of tool development had nearly been reached, and that it is time to give more consideration to the development of the men who have to work the tools and who constitute the heart and brain of the shop.

In regard to the design and construction of tools, the practice of the Pennsylvania R. R. in requiring such tools to be built in accordance with its own specifications was referred to. Some of these specifications were given in our issue of May 12, 1904, in which issue we also discussed the question of improvements in machine tool de-

sign. In one particular case on this road (which may be taken as typical of a number of cases on other roads), through the lack of an appropriation for new shop equipment, it became necessary to make the old tools meet the increased demand due to the increased size and facilities of a new erecting shop and roundhouse. This was done by speeding up the machines and using higher grades of cutting tools. During the experimental work the machines frequently broke down under the severe conditions imposed upon them, and they were repaired and strengthened as required. The various classes of work were then taken in hand by inspectors who checked the time required for each operation, with tools of both carbon steels and high-speed steels. After all these investigations and improvements a standard high-speed steel was selected, the cutting speeds and feeds were determined, and piece prices were adjusted accordingly. The result was a large average reduction in the piece prices, and an increase in wages. The number of engines handled annually has increased from 300 (60 for heavy repairs) under the old arrangement to 1,000 (200 for heavy repairs) under the new arrangement; and the average time spent by an engine in the shops and roundhouse has been reduced from 30 days to 23 days. All this was accomplished without adding new tools to the equipment. It may be contended that such methods are not economical and that better results could be obtained with new and better tools. But it must be borne in mind that in this case (as in many similar cases) the new tools could not be had, while the increased amount of work made it imperatively necessary to make the old equipment meet the new and more severe conditions.

The failure of new shops to at once produce the economies and improvements for which they were designed was referred to by several speakers. An important reason is that the new plant lacks the large equipment of templates, jigs, small tools and handy devices and appliances which have gradually been devised, perfected, and accumulated in an old shop. Besides this, the new tools are often unequipped with the appliances for readily handling the work, and many of the men are new to the work of this particular railway. Not only have the details of shop equipment to be developed, but the working force has to be organized, and both men and machinery "broken in" to the work. It is estimated that the full economy of a new shop can not be realized for at least two years.

We cannot undertake to review or discuss all the various points that were touched upon in the discussion, but enough has been said to indicate the complex character of the problem of how to maintain the motive power equipment in the best condition and with the utmost economy in time and cost. The element of time is of great importance, as it is desirable that the engines should be worked to their full capacity, and while a locomotive is kept in the shop for repairs the investment in it is lying idle.

## CONCERNING EXPANSION BEARINGS IN BRIDGES AND BUILDINGS.

It has become somewhat customary among engineers, in recent years, to criticise expansion bearings for bridges. A large number of stories are current respecting various bridges in which the expansion bearing is said to have rusted fast; and whenever this topic comes up for discussion some of these stories are sure to be brought forward.

At a recent meeting of the Structural Section of the Engineers' Society of Western Pennsylvania, a paper was read in which the author went so far as to condemn the ordinary form of roller bearing and to propose the substitution of sliding bearings at the expansion ends of bridges. The discussion which followed the paper was quite in harmony with the ideas advanced by the author.

We print the paper and discussion in this issue, not because we approve it, but because we are diametrically opposed to the position taken by the author and wish to call attention to the matter so that engineers may be not misled by the statements in the paper. It is unfortunate, we think, that some of the well-known structural engineers in the Society did not present, in the discussion, a defense of the roller bearing, so that the paper might in its final publication be accompanied by some word at least that would tend to put the reader on his guard against an unqualified acceptance of the author's ideas.

In the first place, let us clear away the error that the provision of expansion bearings is a sort of theoretical fad, and that it really makes no particular difference whether provision for free expansion is made or not. The undeniable truth is that the expansion of steel with increasing temperature is just as real and actual a fact as is its stretch under tension; and if the two ends of a rigid steel structure are held fast, cold will produce tension and heat compression in the structure just as surely as if the tension or compression were produced by an external load.

Of course viaduct spans supported on tall, slender columns may move the tops of their supports instead of their expansion bearings; and, in lesser degree, lateral yielding of the supports may occur in nearly every structure when its length is changed by rise or fall of temperature. But it is the engineer's business to know where the expansion and contraction of the structure he builds are going to be taken care of, and it is also his business to be certain that it is done in a way that shall cause no unsafe stresses in the structure or its supports. This much is, in fact, practically admitted by Mr. Rice in the paper before us. If any other engineer is inclined to look upon the provision for free expansion as a mere "theoretical requirement," we suggest that he study the cases of bridges which have suffered injury as a result of their expansion bearings ceasing to operate.

Mr. Rice, however, admits that expansion bearings are needed, but suggests that the roller bearing is a mistake, and plain sliding shoes, such as those used on short-span plate girders would be better practice. He justifies this recommendation by the following, which we quote verbatim:

> Now, as regards certain factors of friction. Roller friction is a very uncertain quantity, and depends so entirely on the finish of the surface of both the beds and the rollers themselves that it is difficult to give any average statement for it. A fair average coefficient would be 0.002 of the weight, which is the stress required to move a body under ordinary rolling friction with such conditions as originally exist in a roller shoe. The sliding friction such as would be occasioned in the first type of shoe, ranges from .07 to 0.11 of the weight for the start, and after the start is made it comes down to as low as 0.004. In other words this sliding friction after the shoe is started is just about twice as much as the rolling friction, if each were in perfect condition.

We may accept Mr. Rice's figure of .002 for rolling friction as probably sufficiently accurate for practical purposes; but where, oh where, is his authority for any such coefficient of sliding friction as 0.07 to 0.11 for the friction of rest and 0.004 for the friction of motion?

Bear in mind that the surfaces he is referring to are unlubricated and will be of necessity more or less covered with the dust and dirt which he claims are so detrimental to the roller bearing. If his figures were correct, an iron weight of 1,000 lbs. resting on an iron plate would be started sliding by a horizontal force of only 70 lbs. to 110 lbs., and once started would be kept in motion by a pull or push of *only four pounds*! It does not require any text-book knowledge to show the absurdity of such statements. Any practical man who has ever moved a heavy weight knows that things are not slid so easily as all that.

The fact is that the coefficient of friction of rest for metal on metal surfaces, unlubricated and dry is from 0.20 to 0.40. If Mr. Rice or any other reader doubts it, he can easily test the matter for himself by simply measuring the angle of inclination of a plane steep enough that weights resting upon it will start to slide.

It is true that the friction of motion is much smaller than the friction of rest; but under any circumstances it is very much larger than the 8 lbs. per ton claimed by Mr. Rice; and, moreover, for our present purpose the friction of *motion* is not at all important. The stress produced in the lower chord of a bridge by temperature movement is dependent on the friction of *rest* of its expansion bearing. Stress enough must be put into the chord to start this bearing, and it is this that concerns the engineer and not the stress that may exist when the bearing has started to move.

This fact is to be borne in mind when considering the possibilities in the use of sliding shoes of bronze or other non-corrosive material, kept oiled with graphite paste, as was proposed by one speaker in discussing Mr. Rice's paper. We know of no figures for the friction of rest of such surfaces which would be at all comparable with the friction of a roller bearing, either at the start when everything is new, or after the structure has been exposed to climatic influences for twenty or thirty years.

Of course it is true that roller bearings are in most cases so designed that water and dust may enter and cause rust, which will interfere with the free movement of the rollers; but that does not strike us as any argument against the roller bearing. It is rather an argument for so designing it in the first place that water and dust will be kept out, and then keeping it in order by proper inspection. Even if all this is not done, and the roller bearing ceases to act, we do not see but that the upper shoe can slide on the rollers so that the roller bearing will become a sliding bearing. Such a condition would not, we believe, be a desirable one, but we fail to see wherein a bridge in that condition would be any worse off than if it had been furnished with sliding bearings in the first place.

We make the above criticisms in the interest of what we believe to be good practice in bridge designing. The points we would emphasize are that provision for free expansion by some form of anti-friction bearing is an absolute necessity of safe construction. If the roller bearing is not satisfactory, try a suspended link bearing or some other plan; but let us not be misled by the idea that ordinary sliding surfaces can fulfil the requirements of an expansion bearing for long-span bridges.

The question of expansion bearings in bridges brings up the kindred question of expansion in structures of the mill-building type: that is, steel-frame mill and shop buildings, train-sheds, and the like. Many structures of this sort are being built 300 ft. and over in length. They are, in the majority of cases, exposed to temperature changes nearly as great as those affecting bridges; and, while the governing conditions (loading, method of support, distribution of steel, etc.) are quite different in the two fields, there are many phases of the expansion problem which are broadly the same for bridges and open mill-building structures.

In the case of bridges it is pretty generally agreed that some provision for expansion is essential when the span exceeds about 50 ft. Mill-building practice does not exhibit such general agreement: there is considerable difference in the views of the expansion problem taken by the various designers and builders concerned in the work. Some do not hesitate to rivet up in one continuous framework a mill-building 300 ft. or more in length. Others plan for expansion-joints as close together as 150 ft., or even closer. Others, again, are half-hearted and doubtful, and provide certain slotted rivet-holes, or certain sliding-plates at girder supports, without working out details that will be sufficiently good to permit any effective expansion motion. Or, the designer and the builder may not agree in their opinion of the need for expansion joints, and therefore may fail to cooperate in producing the result. Thus, we described, not long ago, a steel mill-building where the designer planned a neat and effective expansion-joint to break up a continuous section about 200 ft. long. When the building was fabricated and erected, however, the expansion detail proved to have been changed so that it was the same as the fixed-end detail, and to have been riveted up solid, absolutely preventing the contemplated expansion play.

An unsettled state of opinion, such as this about expansion in mill buildings, is not inherently undesirable or deserving of criticism. It is the natural condition precedent to the settling of a question wherein, as in this case, many elusive factors are involved. And, so far as we know, there is no recorded observation or definitely analyzed experience, to throw light on the need for expansion details in mill buildings. An argument for providing such details may be made on lines very similar to the corresponding argument for bridges. It is true that in buildings there is rarely opportunity for serious stresses in the framework from

temperature expansion, but, if expansion be not provided, the inevitable changes of length will distribute in an indeterminate way, the framework may be racked, sheathing joints loosened, crane-runaways distorted, etc., with results which are sometimes very annoying.

On the other hand, with building supported on columns, a considerable expansion movement can be taken care of in the columns without difficulty, and in all enclosed buildings the annual variations of temperature are much less than in the case of bridges.

What the engineer should do therefore in either bridge or building, is to ascertain just what expansion will take place and make sure that the motion will occur without overstrain or injury to any part of the structure or its supports.

## LETTERS TO THE EDITOR.

### Harlacher's Method of Computing the Discharge of Streams and Canals.

Sir: I wish to acknowledge my error in stating in my recent article on stream measurement (Eng. News, April 6, 1905), that the method I used was more convenient than Harlacher's method. Since writing the article I have used the latter method and find it somewhat shorter, and for general excellence I believe the Harlacher method superior to any other. Respectfully yours,

J. H. Cunningham.

Chamber of Commerce, Portland, Ore., April 15, 1905.

### The Equipment for an Engineer Going to Panama.

Sir: I have read with much interest the letters of your contributors on the equipment for an engineer coming to this country.

After several years' experience in this region I have concluded that the most indispensable article for a young engineer who desires to work here is a fair amount of common sense. The necessary clothes may be obtained at almost any store here after the first pay-day, if they have not been previously provided, but the other article is at a premium. Yours very truly,

H. W. Durham,

Resident Engineer, Dept. Water-Works and Sewers. Ancon, Canal Zone, Isthmus of Panama.

### Imbedding Rusted Steel in Concrete.

Sir: A seven-story reinforced concrete building is being erected here, and I notice nearly all, if not all, of the steel, both in walls and floors, is put in place covered with rust. No oil or other preparation appears to have been used and no effort whatever made to prevent or remove the rust. It is pretty well established, I think, that iron or steel surrounded by good cement or hydraulic mortar will not oxidize, but in important concrete construction it would seem like taking too many chances to put the reinforcing material in the work covered with rust.

I would be glad to know the opinion of engineers and architects on this point, and learn something about the general practice in regard to it.

Very respectfully, Addison M. Scott.

Charleston, W. Va., April 20, 1905.

(The concensus of opinion appears to be that rusted iron or steel imbedded in concrete does not rust more. The cement appears to penetrate through the rust to the clean metal beneath and preserve it from further oxidation. If it were actually dangerous to imbed rusted metal in concrete, then reinforced concrete construction would quickly go out of use; for it is practically impossible to put reinforcing rods in concrete without more or less exposure to the weather and consequent rust.—Ed.).

### The Customary Method of Measuring Sewers.

Sir: In your issue of Apr. 6 you ask information as to the customary method of measuring the length of sewers for estimating payments to contractors. In our experience we have never found a contractor who expected anything different from the sensible method of horizontal measurement. Surveys, plans and profiles are worked out by horizontal measurement, and depths of cuts scaled vertically on profiles. Certainly the quantity of excavation and backfill will be computed that way, and this is the largest item in the cost of construction of the smaller-sized sewers. The only difference between measuring horizontally and on the slope will be in the length of pipe furnished and laid, and we cannot conceive of conditions in practice which would make this difference in length of small sewers worth quibbling about. It would be as sensible to expect the engineer to make a deduction because fifty 2-ft. lengths of sewer pipe will usually lay a little over 100 ft. of sewer.

We find in specifications for sewerage prepared for Plainfield, N. J., and for some dozen other cities and towns, the following:

In making the final estimate of the work the horizontal length of the sewers will be used, without regard to the grade on which they are laid.

It seems to us that any contractor who would expect any different method if that provision were omitted, would be of the objectionable "loop-hole" variety.

Very truly yours, Waring, Chapman & Farquhar.

MacIntyre Building, New York, April 15, 1905.

### Impregnation of Sand and Gravel Deposits with Cement.

Sir: Can any of your readers furnish positive information, or at least practical suggestions concerning the possibility of economically rendering watertight natural deposits such as sand and gravel by impregnating them with some substances, such as cement-grouting, lime-tar, or other suitable material by means of a row of bore-holes driven into said deposits and applying through same the fluid under high pressure?

For the purpose of raising the underground water level in valleys, inclosed by mountains, also in order to separate filter fields for the purification of water running from some manufacturing establishments, sometimes even for improving deficiencies in reservoir sites, this expedient would be very handy, if the cost did not prohibit its application. Very respectfully,

Charles R. Steiner.

U. S. G. S., Chamber of Commerce Building, Denver, Colo., April 4, 1905.

(The late Robert L. Harris, M. Am. Soc. C. E., developed a method of grouting deposits of gravel, sand, etc., in making foundations, and described it in Trans. Am. Soc. C. E., Vol. 24., pp. 230-245, and also in Engineering News of March 14, 1891. W. R. Kinipple, M. Inst. C. E., of London, also developed a similar system, which was described in Engineering News in a series of articles published in the second volume of 1890 and the first of 1891.—Ed.).

### On the Question of Improved Rails.

Sir: Complaints about the inferiority of T-rails as at present manufactured are common. It is also a common claim that as loads have increased so has the section of the rail and everything being equal the rail should wear as well as the older sections. Nevertheless, rails as manufactured at present fail to give satisfaction.

It would be abnormal if they did otherwise. Provision has been made for the increased load by the heavier section but modern conditions demand more. Not only have locomotives and cars increased in weight, but trains have also increased in length and are run more frequently than in former years, so that the rail, subject to modern conditions, has to carry a largely increased wheel mileage as compared with the rails of an earlier period. In consequence of the increased wheel mileage, the metal must fatigue sooner and rail failures are the natural consequence.

Few changes have been made in the specification of the chemical composition of rails. Engineers have mostly followed the old standard specifications, with the exception of such as had the assistance of men who were making a special study of the subject.

The question of an improved rail to better meet modern conditions is not so much one of the external form of the rail as it is one of chemical evolution. It is the solving of a problem in metallurgy, to ascertain by density and crystallization of metal, by chemical proportion and composition, and by improved methods of manufacture the quality of metal which will best resist the wear and tear of modern railroading. Efforts in this direction should be encouraged by the railways in a willingness to pay a better price for a better article.

Respectfully,

G. W. Stadly, Mem. Can. Soc. C. E.,
Const. Engineer Algoma Central Railway.

Sault Ste. Marie, Ont., April 14, 1905.

### Loads on Culverts Under High Embankments.

Sir: With reference to the question raised by Mr. G. R. Winslow in your issue of March 9, 1905, allow me to offer a few remarks:

Under earth and clay-embankments which retain water at times and at other times may stand dry, it would seem to be the part of wisdom to count on the whole load vertically above the horizontal projection of the culvert. Such embankments may form cracks, and no one can tell how these may run. It is true that after slight settlement and consequent reduction of cross-section of the culvert, the pressure would again be reduced on account of the arching action of the load. Whether the culvert constructed for a reduced load could satisfactorily stand such deformation depends partly on the size of the object and partly on its construction and material. One of the principal points to be considered in this connection would certainly be the probable damage to life and property resulting from a total collapse or even from a fracture of the walls.

Numerous portions of tunnels with untold possible tons of load on top have been constructed, which certainly depend for existence on the arching-action of the load. This is proved by the fact that after some reduction in height, they have reached a state of equilibrium. How much of this result is caused by the automatic adaptation of the arch-shape selected to the pressure intensity really acting on top and sides and how much to the arching action of the load will probably in all such cases remain a matter for conjecture, although it is evident in many cases that the arches could not in any shape support the total load without such arching.

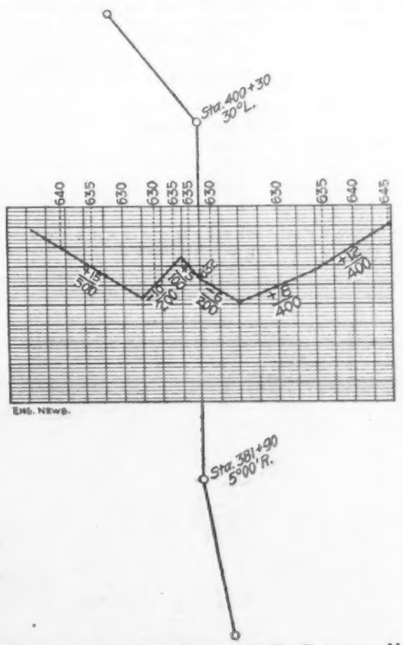
In general it seems clear that a culvert under a high embankment should have an arched roof, or at least one with a rise in the center in order to give the greatest safety. If the case would warrant the expense to add articulations at haunches and center of the arch, there would seem to be a fair chance to count on the self-arching of the earth without apprehensions of collapse. In such an event the writer would deem it reasonable to assume as the load on the articulated arch the earth pressure contained between the two planes tangent to the outside of the masonry and inclined at 45° towards the center (this being the inclination of probable earth rupture in an earth slip).

It should be cautioned, however, that for greater spans than usual in tunnels at present, or when using concrete arches unusually cut down in thickness, the rigidity of the segments between articulations would be another element to be considered. "Assistant Engineer."

Chamber of Commerce Bldg., Denver, Colo., April 16, 1905.

### A Simple Method of Taking and Platting Contours in Preliminary Surveys.

Sir: As the methods of railroad location by means of contours carefully run out during the preliminary survey is now taking the place most everywhere of the original practice of "running the alphabet out" in preliminaries, a method of taking and platting up contours that has been used very effectively might be interesting to many. At the time the preliminary is being made a topographer and one rodman if the distances are simply paced, or two rodmen if measured, take an ordinary hand level and a board or level rod divided into feet and tenths: then standing at points on the preliminary where the slopes change transversely, they take readings perpendicularly to the line of the survey. The rodman is sent out as far




Sketch Showing Position of Profile Paper on Map for Transferring Contour Points.

as the slope is uniform and the distance and the rise + or fall — in feet recorded as a fraction, the distance being the denominator and the rise or fall the numerator. Then the topographer moves up to where his rodman stood and sends the rodman ahead as far as this new slope continues uniform, and so on as far out from the preliminary as contours are desired. Then the same thing is done for the other side of the line, recording the notes from the bottom of the page to the top, as one would be looking

ahead in following the line, recording reading left of the line on the left side of the center line of the topography book and those right of the line on the right side of the center line, as follows, for instance:

| | +15 | −10 | +5 | −6 | +8 | +12 |
|---|---|---|---|---|---|---|
| Sta. 596 | 500 | 200 | 100 | 200 | 400 | 400 |

Note that the distances are all independent of each other, and the sum of the denominators gives the total distance out from the center line. To plat these notes, ascertain from the profile or the level notes the elevation of the point on the center line at which the topography was taken and plat the elevation on a scrap of profile paper and then lay off your first slope, as so much rise or fall in the distance shown by the denominator, then the next one from this point and the same for the rest of them. Then take the profile paper, placing its center line over the center line of the survey on the map drawn to 400 ft. scale to 1 in. and at distances out from center line at which these slope lines drawn intersect the 5 foot elevations, and make a point marking its elevation thereon. In this way all the 5 feet (or smaller) contour points can readily be platted. This same piece of profile paper can be used for four or five miles of contours by making the lines light and erasing each time after the intersections are transferred to the map. Herewith will be found illustrated the method explained, using the notes given in the example of field notes.

From these points of elevation and the profile, the contours can readily be connected up and inked in on the map.

Respectfully,

Jno. E. Walker.

625 New Nelson Building, Kansas City, Mo., March 3, 1905.

## Notes and Queries.

T. M., Dallas, Tex., writes:

A says that a right-hand valve is one which, when closed, you have to turn to the left to open. B says that a right-hand valve is one which, when closed, you are required to turn it to the right to open. Which one has the correct opinion?

We believe a right-hand valve is one in which the hand wheel turns to the right to screw down or close.

We have received the following additions to the list of names of manufacturers of concrete block machines, published in our issue of April 13 and April 20:

Walton Granolithic Stone Machine Co., 18th St. and Wabash Ave., Kansas City, Mo.
W. P. Bryson, 939 N. Y. Life Bldg., Kansas City, Mo.
Diamond Cement Block Co., Rock Rapids, Ia.
Underwood Building Block Machine Co., Fostoria, O.
J. A. Noble, Fostoria, O.

"A Canadian Reader, asks the following question:

Will some reader of the Engineering News kindly inform me what are the determining conditions in choosing between cast-iron and steel columns in buildings?"

Cast-iron columns are usually somewhat cheaper, particularly for short columns, heavily loaded. Practice has been tending toward the use of steel columns more and more in recent years, however, and the rapidly extending use of reinforced concrete is likely to still further crowd out cast-iron columns. Of course, in certain localities, where cast-iron columns can easily be obtained from local foundries while steel columns would have to come from far distant mills, and there are no experienced workers in reinforced concrete, cast-iron columns will continue to be used for a long time to come.

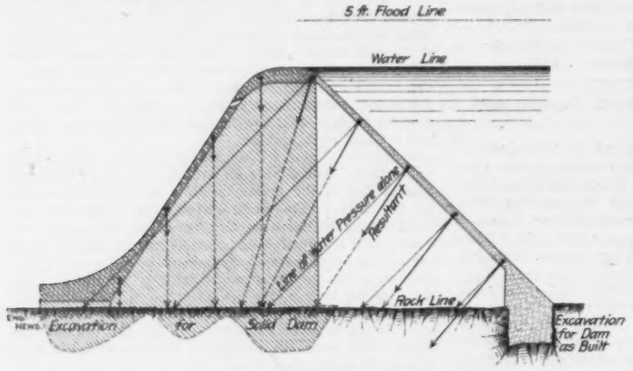

5 ft. Flood Line
Water Line
Line of Water Pressure above Resultant
Line of Water Pressure below Resultant
Rock Line
Excavation for Solid Dam
Excavation for Dam as Built

FIG. 1. COMPARATIVE SECTIONS OF REINFORCED CONCRETE HOLLOW DAM AND ORIGINAL DESIGN FOR SOLID CONCRETE DAM, SCHUYLERVILLE, N. Y.
(The hatched portion represents the original design.)

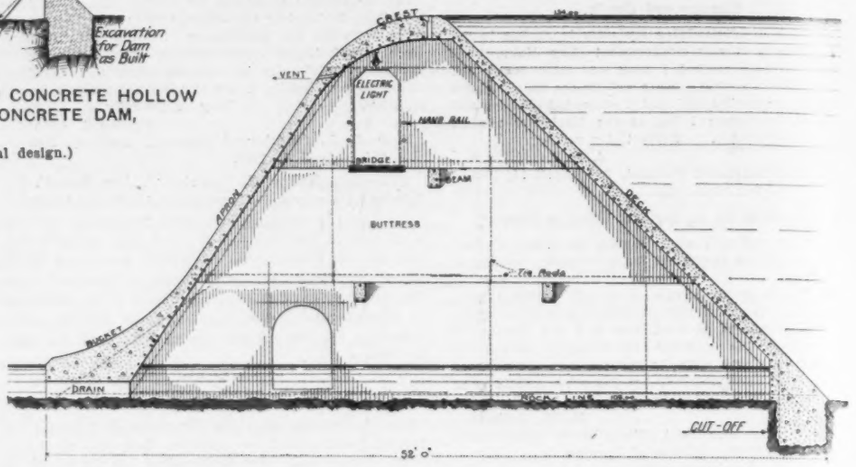



FIG. 2. DETAILED CROSS SECTION OF HOLLOW REINFORCED CONCRETE DAM AT SCHUYLERVILLE, N. Y.

## A HOLLOW REINFORCED CONCRETE DAM AT SCHUYLERVILLE, N. Y.

The dam here illustrated is a reinforced concrete dam under a moderately high head and on a fairly firm rock foundation. The average height of the rollway above the river bed is 25 ft., the maximum height in the channel being 28 ft. The length of rollway between abutments is exactly 250 ft. The foundation is of Hudson River shale, moderately firm in texture, but not sufficiently so to permit a sheer overfall.

A solid concrete dam was originally designed for this location. Alternate designs for the rollway were submitted by the Ambursen Hydraulic Construction Co., and approved by the consulting engineer.

Fig. 1 shows a cross section of the solid dam superimposed on a cross section of the reinforced dam as finally adopted. In this diagram the lines of water pressure alone, normal to the dam, all fall within the base of the dam. When the weight of the dam is calculated and combined with the water pressure the resultant lines as to intensity and direction are as indicated, being more nearly vertical as they approach the crest of the dam. These resultants fall far within the base and indicate the absolute stability of the structure under any flood. The angle of these resultants being greater than the angle of friction between the dam and its base removes the possibility of sliding and does away with a costly excavation for locking walls. A trench 5 ft. wide and 3 ft. deep receives the cut-off wall at the heel of the dam. No excavation was made for the buttresses or toe, the only preparation being to wash the silt and slime out of the crevices of the rock by a jet of water under pressure.

Fig. 2 is cross section of the dam as actually constructed except for a slight modification of the

FIG. 3. SCHUYLERVILLE DAM THREE DAYS AFTER COMMENCING TO SET FORMS.

FIG. 5. FIRST SECTION OF SCHUYLERVILLE DAM NEARLY COMPLETED.

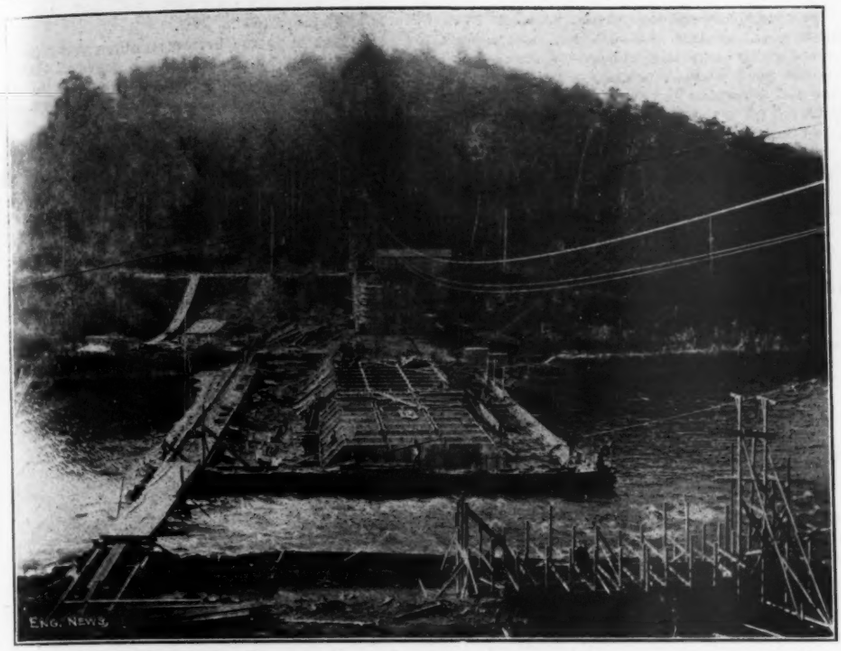


FIG. 4. FIRST SECTION OF THE SCHUYLERVILLE DAM UNDER CONSTRUCTION.

shape of the crest. Openings through the buttresses admit of a free examination of the interior. The dam stands at the head of a slight pitch sufficient to keep it practically drained at ordinary stages of water. During flood, however, the height of the tail water is always the same inside the dam as below it, there being free communication through the drain openings.

The shale foundation has minute seams which with a solid dam might allow sufficient leakage to create an upward pressure on the base of the dam and so impair its stability. With the hollow construction such leaks can exert no pressure since as soon as they pass the cut-off wall they are free.

The 3-in. vent openings shown below the crest in each bay serve to admit air behind the overfall, thus breaking the partial vacuum and preventing "trembling" of the dam.

A unique and thoroughly practical feature is the bridge-way through the dam underneath the crest, the footboards of which are 16 ft. above the river bed. The bridge is dry and well ventilated and lit by an incandescent lamp in each bay. This is the only means of communication between the mill on the north bank and the railway station on the south bank of the river. It is in daily use by all the operatives, officers and visitors who pass and repass, often with 4 or 5 ft. of water going over the roliway above their heads. Two notable features are the absolute quiet in the interior of the dam, notwithstanding the turmoil of the falling water outside, and the powerful indraft of air created by the suction of the vents in the apron and the drains in the toe.

The buttresses in this dam are 8 ft. c. to c., although 10 ft. is the preferred practice. The thickness of the buttresses is 12, 15 and 18 ins. from the top downward for the three benches, respectively. Tie rods are imbedded in the edges and over the opening as shown in Fig. 2. The buttresses are laid in 1:3:6 concrete, using aggregates up to 2½-ins. The maximum load on the buttresses at any point is 5 tons per sq. ft.

The crest is 24 ins. thick in the thickest part, and contains sockets for the flashboards.

The deck* is 9 ins. thick immediately below the crest, increasing to 11 ins. at the bottom. Reinforcement was by Johnson bars, the sizes and spacing of the bars being so proportioned as to give a factor of safety of five under a 5-ft. flood with concrete six months old. Underneath the bars and supporting them is a netting of wire mesh. The deck and apron are laid of 1:2:4 concrete, with aggregates below ¾-in. and as wet as it would work. No special finish was given to the surface and when the pond was filled there was but one leak discovered, just sufficient to barely show detached drops. This leak closed itself naturally in a few days.

*The nomenclature adopted by the designers is here used.—Ed.

The apron* is 8 ins. in uniform thickness between the crest and the bucket.*

The view shown by Fig. 3 was taken on Sept. 30, three days after commencing to set up the forms. Fig. 4 shows the channel of the river with the first bench of the dam under construction. Lateral reinforced concrete beams for stiffening the buttresses are seen near the further shore.

Fig. 5 shows the dam still further advanced with the channel stopped by a second cofferdam and the water diverted through the prepared openings in the dam and seen just beyond the cofferdam.

Fig. 6 shows the completed dam with a 5-ft. flood on the roliway. The night before this view was taken immense fields of heavy ice up to 44 ins. thickness went out, often stranding on the crest to a height of 5 to 6 ft.

Perhaps the most notable feature of this dam is the ingenious method of closing it. The usual high cofferdam around the gap is entirely omitted at a saving of cost of several thousand dollars, with a month's loss of time and all the attendant risks.

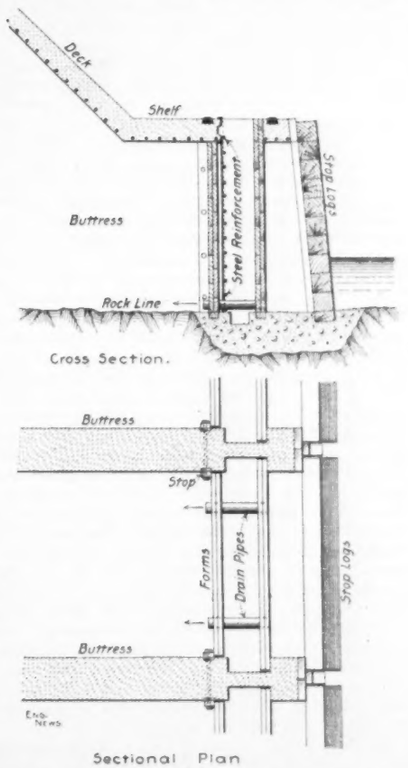



Fig. 7. Temporary Gates for Use at Time of Closure of Dam.
(Patented Feb. 7, 1905.)

The hollow construction of the reinforced concrete dam permits of an entirely unique method of closure. The minimum flow of the river was computed, and also the rate of rise after closing. Enough openings were left, as shown in Fig. 5, to take care of the low water flow when about one-quarter full. The deck and buttresses were extended as shown in plan and vertical section in Fig. 7, to form a horizontal shelf through which openings were left, registering with vertical grooves in the buttresses below. Rough gates of 6-in. timbers were prepared for temporary closing; also two loose fitting forms in each bay. A grid of reinforcing rods wired together at proper intervals was also prepared, the rods being long enough to bear on the concrete shoulders in the buttresses. All this work was done last fall while

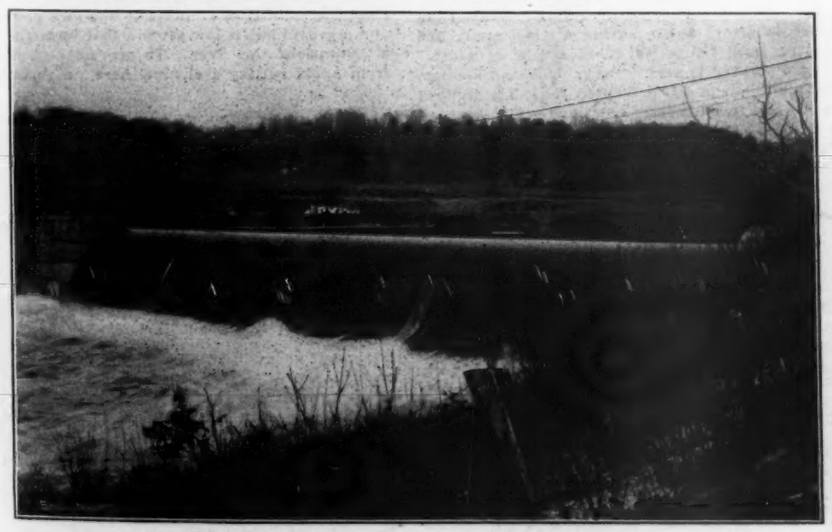


FIG. 6. A 5-FT. FLOOD OVER THE SCHUYLERVILLE DAM.

the main cofferdam was in place, and the material laid in the shelves until required. The flow was then diverted through the openings and the remainder of the dam completed, the last concrete being laid in the crest on Dec. 31, 1904. The forebay at the mill was then completed and the concrete allowed to harden for about one month.

On Saturday, March 11, 1905, the condition of the river being favorable, it was decided to

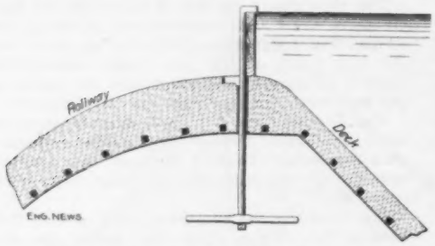



Fig. 8. Design for a Flashboard for Use on Hollow Dams.

close the dam. The ice was first chipped and melted from around the openings. A crew of six men took charge of each opening, all under the direction of Mr. P. M. Trout, Superintendent for the contractors. The water at the time was running about 2 ft. deep through the openings.

One opening was first closed with deliberation, so as to make sure that every contingency was forcseen. A second opening was then closed as the rise of the river was not unduly great. With four openings flowing, the water then rose to a depth of about 3 ft. The four remaining gates were suspended in position by a rope made fast to a temporary gallows frame. At a given signal the ropes were cut and the gates dropped home. A few soft coal ashes were dumped into the stream above the gates for temporarily stopping the leaks. The loose forms were rapidly lowered into place and set out against their temporary stops, the grid of reinforcing rods being at the same time placed in position. Two 3-in. pipe drains were set in holes previously prepared in the bottom edge of each form. At this stage the arrangement had advanced to the condition shown in the cross section, Fig. 7. The space between the forms was then immediately filled with about 1½ cu. yds. of concrete for each bay, and a plank hatch was spiked to nailing strips to protect the green concrete. The full operation of closing the dam occupied a total of 45 minutes from the time of cutting the ropes until the hatches were finally spiked down. One crew of six men on one of the bays completed its work in exactly seven minutes. The particular point in this design is the drain pipes, whose office is to lead away any leakage past the temporary gates which would otherwise accumulate between the gates and the forms, and transmitting the pressure due to the full head as the pond filled up, would of course tend to wash out the cement and ruin the work.

The drains take care of the water with ease and the concrete can harden undisturbed for an indefinite time. When sufficiently hard the drain pipes are then stopped with two soft pine plugs enclosing a plug of concrete. In this case the pond rose to the height of the shelf in about 2½ hours after dropping the gates, thus giving ample time for closing the dam and confirming the calculations of the engineers. The pond itself filled in about 36 hours to the crest of the rollway.

To close the openings left in the apron, flashboards were set on the crest above the openings and for a distance each side and made tight by sand bags. Under the protection of these flashboards, forms were set and the down-stream openings were leisurely closed, this latter operation requiring one full day.

The flashboard and fittings as originally designed for this dam (but which through a misunderstanding were omitted in construction) are another illustration of the possibilities in details when there are "rooms to let" in the interior of a dam. The usual method with flashboards is to set wooden stanchions in a socket and allow them to break off under a flood; or to set light iron rods which can bend over.

The design adopted by the engineers shows a pipe thimble embedded in the crest and open at both ends. In this is a steel rod loose enough to move freely and stiff enough to stand any expected flood without bending. The upper end of the rod is formed as a hook and the lower end is adapted for a two-hand wrench. Fig. 8 shows the flashboards in position. To release the flashboards the rod is turned half way around from underneath the dam and then drawn down until the hook drops into a pocket formed in the concrete. This releases the flashboards, and the rod is out of the way of damage from ice or logs and can be raised at any time and the flashboards replaced.

When the dam has no special bridge-way through it, the slight drip at each rod is not an objection. When, however, there is a bridge-way to be protected the rod is lightly packed with a stuffing box.

The dam was begun Sept. 27, and finished Dec. 31, upwards of two weeks in the meantime having been lost by high water. With the improved methods developed from this experience the engineers state that the construction of a second dam of the same size and characteristics would not again call for more than eight to ten weeks.

The owners of this dam are the American Wood Board Co., Schuylerville, N. Y., Mr. Geo. F. Hardy, 309 Broadway, New York City, was consulting engineer.

The engineers of the rollway and contractors for the steel and forms were the Ambursen Hydraulic Construction Co., 176 Federal St., Boston, Mass. The general contractors for the mill and for the placing of the concrete in the dam, including the construction of the cofferdam, were Messrs. Tucker & Vinton, 156 Fifth Ave., New York. For the information here given we are indebted to Mr. W. L. Church, President of the Ambursen Hydraulic Construction Co.

## CONSTRUCTION, REPAIRS AND SUBSEQUENT PARTIAL DESTRUCTION OF THE ARIZONA CANAL DAM, NEAR PHOENIX, ARIZ.

By H. F. Robinson.*

On account of continued and heavy floods a section of the Arizona Canal Dam, located near Phoenix, Ariz., went out April 13, 1905.

This dam was designed and built under the supervision of the late S. A. Davison for the Arizona Canal Co., to divert water from Salt River for irrigation purposes. It is situated on Salt River, 28 miles above the city of Phoenix and a mile below its junction with the Verde River and forms a portion of the diversion works of the Arizona canal.

This canal, at its head, is 60 ft. wide at the water surface. It has side slopes of ¼ to 1, with a capacity of 1,000 sec. ft. It is 49 miles long, with at least 125 miles of lateral ditches, covering a large tract of land north of Phoenix.

Salt River is an extremely difficult stream from which to divert water, having a wide, sandy bed varying from 700 to 800 ft. to a mile in width.

Its discharge is very erratic. With an average minimum flow of 400 to 500 sec. ft., it has dropped to 100 sec. ft. on several occasions, while floods of from 10,000 to 50,000 sec. ft. frequently occur. One February flood amounted to 285,000 sec. ft., of which 135,000 sec. ft. came from its tributary, the Verde, and 150,000 sec. ft. came from upper Salt River.

With such variations in discharge, and considering the character of the river bed, the design and construction of a safe diversion dam was extremely difficult, since at no place could bed rock for a foundation be found. A rock-filled timber crib dam was designed and constructed which has proven satisfactory in every respect.

At the point selected there is a cropping of rock on either side, though not directly opposite each other; but it was decided to use these rocks as a foundation upon which to anchor the ends of the dam. This necessitated the building of the dam diagonally across the stream, giving a total length of 1,000 ft., as shown in Fig. 1.

The dam was built of timber cribs, of Oregon pine, securely fastened together with drift bolts and tie rods and filled with loose rock and gravel. It was completed in 1887.

At the ends the bottom timbers were fastened to bed rock by anchor bolts. Beyond the rock excavation was made in the river bed in which mud sills were laid and upon which the cribs were built, while for the center of the stream, cribs

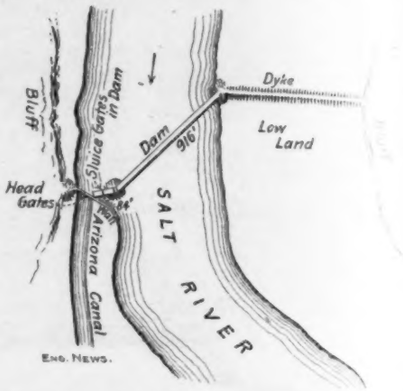



Fig. 1. Sketch Plan of Dam and Head Works, Arizona Canal.

were built, floated into position, filled with rock and sunk.

To provide for unequal settlement the center cribs were not rigidly fastened together, but were attached to each other by wire cable. All cribs were sunk as deep as possible, the maximum depth being 33 ft. near the center of the dam. The base width of the dam, parallel to the course of the stream is from 36 to 48 ft.

The construction throughout the entire length of the dam is not the same. The western half, that nearest the headgates and the portion over which the main current of the river is deflected, consists of a row of cribs planked on the upper side and on the top. (See Fig. 2.) The upstream edge of the top is about 2 ft. lower than the crest edge. On this sloping top and on its upper batter slope, silt has been deposited, adding its weight to the structure.

Below this row of cribs is a second row some 2½ ft. lower; below is a third row with another drop of 2½ ft., and from this the water falls a like distance on to what are virtually swinging cribs that are attached to the dam proper by 1-in. wire cables. The whole of the upper and face surfaces of these cribs are planked over, forming a series of steps upon which the water falls, and its force is thus broken.

The western half is built without these steps and consists of the main cribs with a single apron below, as shown by the full lines in Fig. 3.

Beyond the east end of the dam and separated from the main river channel by the rock cropping and a gravel bar is low ground that formerly was a channel of the river. To prevent high water from again cutting a channel here, an earth em-

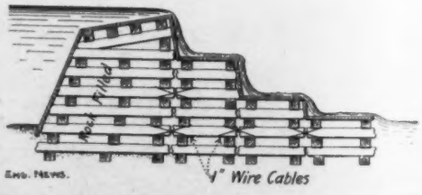



Fig. 2. Cross-Section of Western Half of Arizona Canal Dam.

bankment about 10 ft. high and 1,200 ft. long was constructed.

Since the dam was built the central portion has settled at various times, the most serious occasion being in 1891 when not only considerable settlement took place, but portions of the dam were washed away, necessitating very extensive repairs. In every case prompt repairs have been made by stripping off the covering, building up the cribs to grade, filling them with rock and replanking.

*Irrigation Engineer; City Engineer of Phoenix, Ariz.

About six or seven years ago it was discovered that the timbers in the upper portion, which in low water were exposed to the air, were beginning to decay and it was decided to strengthen the dam and at the same time to raise its crest to a uniform height of 13 ft., or 2 ft. higher than it was before.

Four rows of 12 × 12-in. sawed piling were driven the entire length of the dam, about 5 ft. c. to c., as shown by the broken lines in Fig. 3. On these piles 12 × 12-in. caps were fastened and stringer of like dimensions were placed above these longitudinally, the cribbing thus formed being filled with rock and the whole recovered with 3-in. plank. A row of sheet piling was also driven along the upper side. No repairs have been made since, and none had been needed until the recent failure.

At the time the break occurred there was 7.7 ft. of water on the crest. It was first noticed

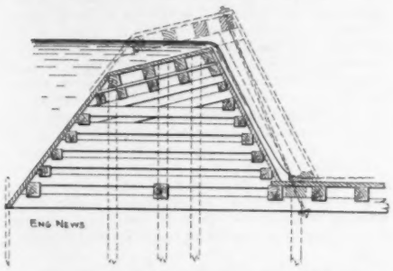



Fig. 3. Eastern Half of Arizona Canal Dam; Showing Also, by Broken Lines, Repairs and Enlargements to Whole Dam.

that portions of the center were being torn away a plank or a timber at a time, and at the present time there is a gap of about 300 ft. The river has fallen to about 4 ft. on the crest and the danger of the destruction of the balance of the dam seems possibly over.

THE THEBES BRIDGE over the Mississippi River at Thebes, Ill., was opened to traffic on April 18. The last pins of the cantilever channel span were driven on April 1.

CREOSOTING WOOD is generally supposed to make it more inflammable. We are informed, however, that several creosoted telephone poles in the recent fire at the New Orleans docks, resisted the attack of the flames successfully. Two of these poles stood within 6 ins. of one of the freight warehouses, so close that holes had to be cut in the edge of the roof when the poles were erected. The warehouse with its contents including 15,000 bales of cotton, was totally destroyed; but the poles were so little charred that they are still in use.

THE NEW ORLEANS FREIGHT TERMINALS of the Illinois Central R. R., at what are known as the Stuyvesant Docks, which were destroyed by fire a few weeks ago, are now being rebuilt on improved lines, and the work contemplated is as follows: 1, The reconstructing of wharf, 3,000 ft. long and 152 ft. wide; the lower portion is a wooden structure, covered by a steel roof. 2, The reconstruction of two freight houses, 100 × 1,200 ft. and 100 × 1,000 ft.; both of these will be of fireproof construction, having brick walls, concrete floors and steel and concrete roofs; 18-in. brick fire walls will be placed at intervals of 240 ft. 3. The reconstruction of the cotton warehouse, about 1,600 ft. long and having an average width of 140 ft.; this will be of fireproof construction, similar to the freight houses. 4. The reconstruction of two grain elevators, having a combined capacity of 2,500,000 bushels. 5. The reconstruction of conveyors, including all machinery, etc. 6. The reconstruction of the special fire protection system. The work will be under the direction of Mr. R. E. Gaut, Engineer of Bridges; and the contracts for the work have been awarded.

DUPLEX LOCOMOTIVES on the Mallet system have been built for the American Railway Co., which operates 160 miles of meter-gage lines in Porto Rico. The engines are four-cylinder compounds and have twelve driving wheels, arranged in two groups. The three rear axles are mounted in short main frames carrying the firebox and cab and extending half way along the boiler barrel. At the front end of these frames are the high pressure cylinders, taking steam from pipes leading up outside of the boiler to the dome. The three forward axles are mounted in a swiveling truck frame pivoted to a cross frame at the ends of the main frames. At the front end of the truck are low-pressure cylinders. The engines have the Walschaert valve gear, operating slides. They burn soft coal, and in addition to long smokeboxes have balloon smokestacks with spark arresters. The tenders have two four-wheel trucks with 26-in. wheels. The engines were built by the Baldwin Locomotive Works, and their dimensions are as follows:

| | |
|---|---|
| Driving wheels (12) | 3 ft. 1 in. |
| Driving wheelbase (of each group) | 6 " 10 " |
| Wheelbase of engine | 20 " 4 " |
| Wheelbase of engine and tender | 42 " 8 " |
| Weight of engine | 106,650 lbs. |
| Weight of engine and tender | 156,000 " |
| Cylinders | 12½ × 20 and 19 × 20 ins. |
| Boiler, diameter | 4 ft. 6 ins. |
| Working steam pressure | 200 lbs. |
| Firebox (radial top) | 8 ft. × 2 ft. 3¼ ins. |
| Tubes, No., 155; diameter | 2 ins. |
| Tubes, length | 15 ft. 6 ins. |
| Heating surface, tubes | 1,251 sq. ft. |
| Heating surface, total | 1,357 " " |
| Grate area | 18 " " |

THE RESISTANCE OF PILES TO ATTACKS OF teredoes (*Nausitoria*) was tested by driving two piles at Port Hunter, New South Wales. Two kinds of hardwood piles were driven in 1897, one of Jarrah timber (Western Australia) and one New South Wales Turpentine (*Syncarpia laurifola*), both being driven in 10 ft. of water where the range of tides is 5½ ft. at spring tides and 3½ ft. at neap tides. In Feb., 1904, 6½ years after driving, the two piles were pulled and sections cut. It was found that in the Turpentine pile the attacks of the teredo had been confined to about an inch of the sapwood, the rest of the pile being as sound as the day it was driven. The Jarrah pile, on the contrary, was completely riddled between high and low water marks. Below low-water mark, both piles were practically sound, and beneath the ground line they were in perfect condition. It is said that this test adds another to the long list of proofs that the New South Wales Turpentine timber is impregnable to the attacks of marine borers under almost all conditions.

The foregoing facts have been abstracted from a report of Mr. J. Davis, M. Inst. C. E., Under Secretary for Public Works, New South Wales.

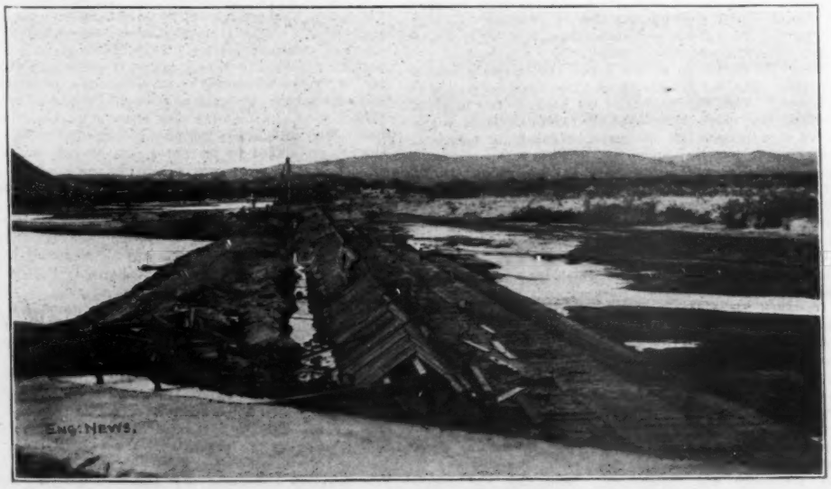



FIG. 4. ARIZONA CANAL DAM BEING REPAIRED AND RAISED.

## PERSONAL.

Mr. Bryce R. Blair has been reelected City Engineer of Carbondale, Pa.

Mr. Harvey Linton has been reappointed City Engineer of Altoona, Pa.

Mr. A. E. McIntosh has been appointed City Engineer of Pueblo, Colo.

Mr. J. P. Harper has been elected City Engineer of Pine Bluff, Ark.

Mr. G. P. Coleman has been reappointed City Engineer of Winona, Minn.

Mr. John B. Hawley has been reappointed City Engineer of Fort Worth, Tex.

Mr. A. P. Smith has been reappointed City Engineer of North Tonawanda, N. Y.

Mr. William F. Williams has been reappointed City Engineer of New Bedford, Mass.

Mr. W. S. Kinnear, Chief Engineer of the Michigan Central Ry., has been appointed Assistant General Manager.

Mr. W. K. Walker has been appointed Division Engineer of the Missouri Pacific Ry., with headquarters at Pueblo, Colo.

George N. Merrill & Co., Consulting Engineers of Springfield, Mass, have been appointed Town Engineers of West Springfield.

Mr. I. Wayne Von Leer, M. Am. Soc. C. E., has been appointed General Manager of the El Carmen Copper Co., at El Carmen, Durango, Mex.

Mr. Geo. A. McCarthy has been appointed Engineer Maintenance of Way and Assistant to the Chief Engineer of the Temiskaming & Northern Ontario R. R.

Mr. F. H. Jones, formerly Manager of the Air Compressor Department of the International Steam Pump Co., has been appointed General Sales Manager of that company.

Mr. Joseph M. Patterson has been appointed Commissioner of Public Works of Chicago, Ill., to succeed Mr. F. W. Blocki, who, at the last election, was elected City Treasurer.

Mr. Oscar Sanne, M. West. Soc. E., recently with the Scherzer Rolling Lift Bridge Co., has gone into business as a consulting bridge engineer. His address is 265 Hazel Ave., Chicago.

Mr. W. H. Hunter, Chief Engineer of the Manchester Ship Canal, has been selected by the British Government to act as a member of the board of advisory engineers of the Isthmian Canal Commission.

Mr. W. S. Kirby has been appointed Superintendent of the Chicago Division of the Chicago, Burlington & Quincy Ry., succeeding Mr. H. G. Hetzler, who has resigned to go to the Metropolitan Elevated Ry.

Mr. Robert R. Livingston, Mechanical Engineer, formerly with the Allis-Chalmers Co., in its Chicago office, has associated himself with Mr. W. S. Barstow, Consulting Engineer, 56 Pine St., New York.

Mr. Charles Ducas, J. M. Am. Soc. M. E., formerly Editor of the Railroad Gazette has become associated with Mr. Ray D. Lillibridge in the business of Technical Publicity, with office at 170 Broadway, New York.

Mr. R. E. Spencer Geare, formerly Consulting Engineer to Colgate & Co., has severed his connections with that firm and will be identified with Mr. Walter S. Timmis, Consulting Engineer, 150 Nassau St., New York.

Mr. T. H. Clark has been appointed General Superintendent of Motive Power of the Chicago, Burlington & Quincy Ry.; and Mr. F. A. Torrey succeeds him as Superintendent of Motive Power of Lines East of the Missouri River.

Mr. Charles J. Poetsch, City Engineer of Milwaukee, Wis., has been reelected President of the Board of Public Works of that city. Mr. Poetsch has been connected with the engineering department of the city for 27 years, having been appointed Assistant City Engineer under Mr. H. F. Hilbert in 1878.

Mr. Arthur A. Reimer has been appointed Superintendent of Water-works of East Orange, N. J. Mr. Reimer was graduated from Massachusetts Institute of Technology in the Class of 1900. He has been Chief Engineer of the Jersey City, Hoboken and Paterson Division of the North Jersey Street Railway. He was field engineer during the construction of the new water-works of East Orange.

Mr. F. H. Newell, M. Am. Soc. C. E., Chief Engineer of the U. S. Reclamation Service has been appointed a member of a special commission which is to deal with questions arising relative to the division and interference with the course of international rivers. The other members of the Commission are Judge Penfield, Solicitor for the State Department and Special Assistant Attorney General M. C. Burch.

Mr. Charles N. Wilson, formerly Manager of the Columbus, Greenburg & Richmond Traction Co., and prior to that fiscal representative of the Appleyard system of traction lines in Ohio, has been elected President and General Manager of the American Engineering Co., a corporation recently organized for the handling of rail-

road and other construction contracts. Mr. J. C. Ostrup, M. Am. Soc. C. E., of New York, has been elected Chief Engineer. The headquarters of the company are in the Traction Terminal Building, Indianapolis, Ind.

Mr. William E. Swift, Assoc. M. Am. C. E., has resigned his position as Engineer, U. S. Reclamation Service, to accept a position as Resident Engineer with the Isthmian Canal Commission. Mr. Swift, who was born in Connecticut, was graduated from the Massachusetts Institute of Technology with the degree of B. S. He has been engaged in various engineering capacities since 1895 with the Boston Water-works, Metropolitan Water-works, Boston; Munson Steamship Line, New York; Rapid Transit Railroad, New York, on construction of subway.

### Obituary.

William Toone St. Auburn, a civil engineer of Detroit, Mich., died April 15, at St. Mary's Hospital, in that city, aged 48 years. He had been employed as a civil engineer in South Africa and New Zealand.

George N. Wheelock, Assistant Engineer in the Bureau of Engineering, Department of Public Works, Chicago, died April 19, at the age of 47. He had been connected with the Bureau of Engineering since 1889.

## ENGINEERING SOCIETIES.

### COMING MEETINGS.

INTERNATIONAL RAILWAY CONGRESS.
May 3 to 15, 1905, at Washington, D. C. Secy. of American Committee, W. F. Allen, 24 Park Place, New York.

AMERICAN WATER-WORKS ASSOCIATION.
May 8 to 12, 1905. Annual meeting at West Baden, Ind. Secy., J. M. Diven, Charleston, S. C.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
May 16, 1905. Annual meeting at New York, N. Y.

ASSOCIATION OF RAILWAY TELEGRAPH SUPERINTENDENTS.
May 17, 18. Annual meeting, at Chattanooga, Tenn. Secy., P. W. Drew, Wisconsin Central Ry., Milwaukee, Wis.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 6, 7, 8, 9, 1905. Annual spring convention, at Scranton, Pa. Secy., F. R. Hutton, 12 West 31st St., New York, N. Y.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
June 6 to 9, 1905. Annual convention in New York. Secy., Richard Moldenke, Watchung, N. J.

{MASTER CAR BUILDERS' ASSOCIATION.
{AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 14-21, 1905. Annual meeting at Manhattan Beach, N. Y. Secy., J. W. Taylor, The Rookery, Chicago, Ill.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 19 to 23, 1905. Annual convention at Asheville, N. C. Secy., R. W. Pope, 95 Liberty St., New York, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 20 to 23. Annual convention at Cleveland, Ohio. Secy., Charles Warren Hunt, 220 West 57th St., New York, N. Y.

LEAGUE OF AMERICAN MUNICIPALITIES.
August 23 to 25. Annual meeting at Toledo, O. Secy., J. McVicar, Des Moines, Ia.

ROADMASTERS & MAINTENANCE OF WAY ASSOCIATION.
Sept. 12, 14. Annual meeting, at Niagara Falls, N. Y. Secy., C. E. Jones, C., B. & Q. Ry., Beardstown, Ill.

ST. LOUIS RAILWAY CLUB.—At the meeting on April 14, Mr. F. Wyman, Postmaster, presented a paper on "The Postal Service in St. Louis." The following officers were elected: President, John J. Baulch; Secretary, E. A. Chenery.

SOCIETY OF CHEMICAL INDUSTRY.—An interesting subject came before the New York section of this society at its meeting of Wednesday, April 19, held at the Chemists' Club, 108 West 55th St., New York City. Mr. M. Toch read a paper entitled "New Paint Conditions as Indicated in the Subway of New York." Beginning by laying stress on the fact that the materials used in painting the steelwork of the New York Rapid Transit subway were excellent and most carefully inspected, and the workmanship as good as the unfavorable conditions would permit, the author said that, in little less than two years after the work was painted, hair-line cracks and pittings appeared on the surface. The steelwork had the following paint treatment: one coat red-lead in linseed oil in the shop, one coat red-lead in oil after erection which was applied over a thoroughly sand-blasted and wire-brushed surface, and finally a finishing coat of white-lead in linseed oil. The treatment was first class, as far as paint knowledge went at that time. Yet the deterioration indicated by the pittings and cracks increased. By a knife-blade inserted in a crack, a large scale or sheet of the paint with an underlying coat of rust could be removed. As samples shown by the author evidenced, these sheets are hard, fairly strong and somewhat elastic, and of considerable thickness even if the paint thickness be deducted. An examination of the rust under the paint showed a surprising condition. The rust differed from the conventional form of iron rust: hydrated oxide. It comprised various distinct layers; the layer nearest the iron showed a large proportion of a blackish material, which proved to be approximate magnetic oxide ($Fe_3 O_4$) in composition, and carried varying percentages of water—nearest the paint, however, the composition and appearance showed that the rust approximated ordinary rust ($Fe_2 O_3$ + water) but the water percentage was variable and differed from that usually considered characteristic of rust. Later, the author found a rust of very similar character to the subway rust on the supporting framework of a steel pier at Atlantic City, N. J. The rust in both cases, moreover, had a marked laminated structure, that is, it presented a crude mechanical lamination besides merely the change in chemical composition from the inner to the outer surface. There is some probability, though this matter has not yet been definitely shown, that the rust sheet contains some laminae of unchanged iron as if the process of corrosion had found some layers of metal more resistant than others.

The cause of the relatively rapid rusting of the subway steel was found by the author in the large amount of moisture to which the framework is exposed, due to the fact that in winter the iron is cooled (by conduction from the street surface downward) below the temperature of the air in the subway. This condition, together with the normally high moisture in the subway, especially during the construction period when the painting was done, caused the linseed oil of the paint to go through a different change than its normal oxidation in the open air. The author holds the process (under the conditions of the subway) to be one of saponification of the linseed oil. His conclusion is that, for the conditions there prevailing, a paint must be used which contains no linseed oil and no other saponifiable material.

The discussion of the paper, which was of much interest, tended to support the author's observations as to the character and arrangement of the rust. It was also brought out that a paint is now being used in the subway which is free from linseed oil and which up to the present is very satisfactory. We hope to present a fuller abstract of the paper and its discussion in a later issue.

Two other papers read at this meeting were: "The Determination of Acetic Acid in White Lead," by Mr. G. W. Thompson, describing a new method which has proved satisfactory in the laboratory of the National Lead Co., and "Analysis of Senna," by Mr. R. W. Moore.

WESTERN SOCIETY OF ENGINEERS.—At the meeting held at the society's rooms in Chicago on April 19, Mr. Richard T. Fox, General Manager of the Citizens' Street Cleaning Bureau, presented a paper covering the work of the first nine months operation of the Bureau. This Bureau was organized by the Commercial Club and the Merchants' Club, to clean the streets in the business center of the city, owing to the conspicuous failure of the city authorities to perform this work. Mr. Fox, who had had experience in the New York street cleaning department, was put in charge of the work, organizing a uniformed force and superintending the operations. All the work is done by hand, using brooms on the brick and granite pavements, and scrapers and brooms on asphalt pavements. The hand work is much more effective than machines would be, partly because the sweeping is done from the center of the street to the sides, thus cleaning out the hollows between the rows of paving blocks. The pavements are also so uneven, that machines would simply sweep a large proportion of the dirt into the hollows and depressions and leave it there. The hand sweeping can also be done by day, when it is most needed. For streets with busy traffic Mr. Fox thought it bad practice to let the dirt accumulate on the surface all day and then remove it all at night.

The work is largely hampered by the local custom of sweeping the dirt from stores and sidewalks into the streets, throwing paper in the streets, piling ashes from boiler rooms in the gutters until such time as they may be carted away, and occupying large areas of the streets with piles of sand, lime and building material wherever building operations are in progress. During the past winter the streets have been kept well cleared after the numerous snow storms.

After the discussion of this paper, the secretary read a paper by Mr. John W. Alvord on "The Paving Problem of Chicago." This showed that in the present state of civilization, the rough granite block pavement is in many ways objectionable, its one merit of durability being far outweighed by its numerous objectionable features, among which are its unevenness and the rough jarring of vehicles over it. Such a city as Chicago should adopt the policy of laying better and more satisfactory pavements, even if they are more expensive and less durable than granite. The evil practice of tearing up the streets for repairs to pipes, conduits, etc., was commented upon; as well as the inefficient way in which the backfilling and repaving is done after such work.

On April 21, a large number of members and guests including ladies) visited the Armour Institute of Technology, by special invitation of the trustees, the president and the faculty. An informal reception was held in the library from 8 to 9 p. m., and addresses were made by the President, Dr. Gunsaulus, on behalf of the Institute, and by Mr. N. C. Ricker (Dean of the College of Engineering, University of Illinois), on behalf of the Society. After the addresses, the party visited the class rooms, laboratories and workshops under the guidance of the professors and instructors. Both day and night classes are held, the latter being for the benefit of men and boys who have to work during the day. The courses include civil, mechanical and electrical engineering, chemistry, building construction, architectural and mechanical drawing, and practical shop work (wood working shop, machine shop, foundry and blacksmith shop). There are also special courses in telephone and fire protection engineering. The shops and laboratories are well equipped, but new machines and appliances are being introduced to meet the increased requirements and to keep the equipment up to date. After the visit of inspection, refreshments were served in the lunch room of the club building on the athletic grounds.

WESTERN RAILWAY CLUB.—At the meeting held in the Auditorium, Chicago, on April 19, Mr. E. F. Tratman (Associate Editor of Engineering News), presented a paper on "Tank Locomotives." The advantages and disadvantages of this type of engine were discussed both in regard to regular road service and suburban, or switching and pusher service. It was shown that in Europe tank locomotives are largely used for all the classes of service, with runs as long as 75 miles, and speeds of 60 to 70 miles an hour. Even in Germany, where railway operation is conservative and under government control, certain classes of tank engines are allowed to run in either direction at speeds of 65 miles an hour. On the whole there seems to be little evidence that a properly designed tank locomotive is unsafe for high-speed service. The principal objection to tank locomotives in this country is that they cannot be used for very long runs, and there is a tendency to employ engines which can be used interchangeably in all kinds of service. This results, however, in light trains on short runs being frequently handled by engines of much greater power and weight than are necessary. Where the suburban or local traffic is heavy, it is worth considering whether it might not be more economical and satisfactory to employ a class of engines specially adapted to the traffic.

The paper described the early introduction of the Forney type of tank locomotive, and also the recent introduction of tank locomotives of various types for the suburban service of a number of railways in this country. Several of these modern tank engines were described in Engineering News of Feb. 16. In addition to this, however, engines for heavy coal traffic and for operation on steep grades were described, including one of 120 tons for the Kanawha & Michigan Ry. This is of the 2-8-2 type, for coal train and heavy switching service, operating on grades as to 4 and 4½%; it has cylinders 19 × 26 ins., and its eight 49-in. driving wheels carry a weight of 92 tons. It carries no less than 5 tons of coal and 2,500 gallons of water.

In regard to foreign tank locomotives, the paper described a number of types used in suburban and regular road service (passenger and freight) on European railways. The largest is one of the Atlantic or 4-4-2 type on the Great Western Ry., of England; this has cylinders 18 × 30 ins., and driving wheels, 80½ ins. diameter. Besides these, a variety of small and large tank engines used in all parts of the world were described, the object being to show the number of types and the various applications of this class of engine. Some new tank engines for the suburban service of Paris were designed to take the place of engines with tenders, owing to the trouble and delay incurred in turning these engines at the busy city terminal; they carry as much coal and water as did the tenders of the old engines. In a number of instances very large tank engines are used for pusher service, and among these are some 94-ton rack-rail engines for South Africa, helping trains up a 5% grade 3½ miles long. They are of the 4-6-4 type, with two inside cylinders for the cog wheels and two outside cylinder for the adhesion wheels.

In opening the discussion, Mr. P. H. Peck, of the Chicago & Western Indiana Ry., spoke of the satisfactory performance of the tank locomotives introduced on that road for suburban service. They are of the 2-6-2 type, with side tanks, and have a run of 11 miles with 15 stops. Mr. W. Forsyth referred to the respective merits of side and rear tanks; for large engines he considered that the former were preferable as they allow of a shorter wheelbase. The satisfactory performance of some of the large engines mentioned in the paper was brought out in the discussion, and there was a general appreciation of the advantages of tank locomotives for certain classes of service.

The report of the committee on "Revision of the Rules of Interchange" was then taken up, and the proposed changes were approved. These will be submitted for discussion at the meeting of the Master Car Builders Association in June. The most important changes were in regard to the limits of wear of wheels, classifying them for cars of less than 80,000 lbs. capacity and of 80,000 lbs. or over; allowing center sills to be spliced, which is not permitted under the present rules.

A paper on "Electricity on Steam Railways" was presented by Mr. C. F. Street, but as members had not had much time to read the advance copies and as the hour was already late, it was voted to postpone the further consideration of the paper until the next meeting, May 16.

The secretary announced that the club has now a total membership of 1,133 members.

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